AD-A131 122 TECHNICAL FEASIBILITY STUDY OF NAVY PIER CONCEPTS CONCEPT 2 THE FLOATING..(U) LIN (T Y) INTERNATIONAL SAN FRANCISCO CA APR 83 NAVY PIER CONCEPTS-2/83 1/1

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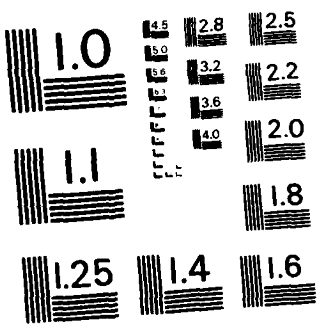

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

2

NAVY PIER CONCEPTS
REPORT No. 2/83

AD A131122

THE FLOATING MARINA PIER

SUBMITTED TO:

DEPARTMENT OF THE NAVY

OFFICE OF NAVAL RESEARCH
ARLINGTON, VIRGINIA

SUBMITTED BY:

T.Y. LIN INTERNATIONAL

APRIL 1983

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

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18. SUPPLEMENTARY NOTES

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Ports	Design Feasibility	Marina Pier
Piers	Floating Pier	Concrete Piers

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The concept of a floating marina pier is presented. This pier concept offers the benefits of off-site construction, less critical waterfront site selection, tidal independence, and reduced fendering requirements. Disadvantages include high initial cost, increased inspection requirements during service, and a vulnerability to sinking. Design challenges are seen for the connection between modules, anchoring systems, service distribution, and the shore-pier interface.

THE FLOATING MARINA PIER

1. INTRODUCTION

The floating marina pier which was developed and presented to ONR in September, 1981, is the second of three concepts selected for further study under the contract for 1982/83. Interestingly, all three concepts of the future Navy pier are common in their basic features of flotation and mobility. The difference between them is only in their application. The emergence of a floating and mobile pier concept is perhaps a logical next stage in the evolution of a Navy pier, since the fixed position and fixed level pier have been with us for a long time, and any improvement thereon at this time would at best be incremental and not generic in nature. Increasing the working area of a deck by adding another deck on the fixed pier can also be considered an improvement in the former category.

The floating pier will solve the traditional problems of a fixed pier — congestion on deck, slow turnaround time, inadequate services, and heavy maintenance costs, particularly from the replacement of the camel fendering system.

Although a floating pier is new to the Navy, the state of the art pertaining to the design and construction of a floating vessel is not. There are numerous instances and experiences that have proven the feasibility of a floating vessel being used as a pier. The Navy itself has built several prestressed concrete floating piers measuring 500' x 100' to serve as docks for seaplanes in the early 50's, in Honolulu, Alameda and San Diego. Several concrete floating dry docks were also built in WW II, and these had operated continuously for forty years. Recent examples of concrete vessels include the 65,000-ton LPG barge for ARCO that measures 136' wide by 460' long, now operating off Indonesia, and the floating pier terminal measuring 30' x 100' x 700', which is under construction in Tacoma, Washington, for service in Valdez, Alaska.

The format of this report will be similar to the first one on the floating expeditionary pier. It will investigate the general validity of the design, etc., that were used in the development of the concept in 1981, study the feasibility of materials and construction methods, and identify problem areas.

2. THE ADVANTAGES OF THE FLOATING MARINA PIER

The advantages of a floating pier for Naval applications have been brought out and discussed in published materials, particularly the study on floating piers submitted by T.Y. Lin International to the Naval Civil Engineering Lab in Port Hueneme in 1982. For the sake of the completeness of this report, the major advantages described in the study may be summarized as follows:

a) The pier can be constructed offsite, then towed to site for installation when completed. The advantages here are the reduction of construction time, cost, and traffic congestion at the site.

b) A floating pier, particularly one that can be extended far out into deeper waters, does not take up as much valuable waterfront real estate as a fixed pier. It is also not as demanding as a fixed pier in the selection of a suitable site.

c) By being able to move up and down with the changes of water level, it will avoid the customary and considerable changes in the length of mooring lines, brows, hoses, electric cables and other utility lines that connect the pier with the ship, thus reducing the wear and tear, as well as the knotting and the pinching of these lines due to excessive coiling and stretching that are common with the fixed pier.

d) The near constant pier/ship level realtionship will make it possible to use fixed position modern fendering systems, instead of the traditional camel fendering system which is subjected to heavy maintenance problems and costs.

e) It can provide a second deck without much additional cost. This advantage has been recognized, as evidenced by the double-deck pier being considered for Charleston Naval Station. For the floating pier, the addition of the second deck is almost "natural." This is because of the available space between the main deck, which is located 16' above the waterline, and the buoyancy chamber, which is below the waterline. This space serves no purpose other than to support the main deck and could easily be turned into a second deck with little additional effort and cost. The second deck will go far in relieving congestion, not only in traffic but also in the utility pipelines, and will perform additional functions such as providing space for training facilities, workshops, and parking. It will undoubtedly increase the efficiency of the pier and heighten the combat readiness of the docked ships.

f) It will increase cost effectiveness of the pier by making it reuseable. Thus, a floating pier that has become obsolete in one location need not be disposed of by demolition, but could be relocated at another site where it could be useful.

g) Its mobility and relocatability will enhance the deterrent effect of the Navy in general. To maximize this effect, the pier should be located in a relatively large body of inland or sheltered water. There is a number of such waters around the USA, e.g., the Great Lakes that feed into the Hudson River, Puget Sound in Oregon, the San Francisco Bay, the Sacramento Delta area in California, and the Galveston Bay in Texas.

h) The marina pier concept will cater well to the future expanded Navy because it can be modularized, mass-produced and made up to almost any size within physical limits by assembling the number of modules required. The major limitation is the capacity of the anchoring system and the incidence and magnitude of wave and seiche action at the site.

i) Better earthquake resistance can be expected in a floating pier.

There are, of course, also disadvantages to this concept. The first would be its vulnerability to blockage if the ships' access to the sea is cut by enemy action. This would be true for the traditional Navy ships. However, for the marina pier operating in the future, the Navy ships it was to cater for and as projected in the original concept, have been envisioned to be vessels capable of skimming over the water surface at great speed like a hydrofoil. The blockage of the waterways would have to be fairly complete before vessels of this type could be stopped.

Other disadvantages exist with the floating pier concept which must be considered in a final evaluation. Some of these are: high initial cost, as high quality control during construction and larger quantities of materials are required; more inspection to ensure seaworthiness is necessary; vulnerability to accidental sinking and relative movement at the pier-shore interface which may pose some problem to the design.

3. DESIGN CONSIDERATIONS

The design parameters that have been established for the marina pier are as follows:

a) Dead and live loads. Refer to Appendix A, Sheet A-1.0 for loading criteria.

b) Wave loads according to Sea State 2; e.g., a wave height of 3' maximum and a wave length equal to the pier length parallel to the direction of the incident wave. The design procedure for wave loading follows that of the ABS rules and classing of vessels.

c) Wind loads according to Sea State 2; e.g., a maximum wind velocity of 50 mph. The wind forces and their distribution are done according to the NAVFAC's Design Manual DM 26.

d) Current of 3 knots at water surface decreasing linearly to zero at the water bottom and constant current velocity from pier end to shore. The current loadings are estimated using the new Navy DM 26-6 method. Wave, wind and current forces are assumed to be acting concurrently when full berthing at the floating pier (Scheme 2) occurs.

e) Double deck spine pier only.

f) Safety considerations.

g) Modular construction.

h) Maximum draft of 27.5' due to dead and full live load using hard rock concrete. Water depth of 40'. If lightweight concrete is used, the draft will be reduced to 24.5', thereby extending the usefulness of the pier to more shallow sites.

i) Self-sufficiency.

4. DESCRIPTION OF THE PIER

Two marina pier schemes, each with 12-berth capacity, are presented. Scheme 1 is similar to the original concept of 1981 that caters for the projected future Navy ships that can speed like a hydrofoil, and maneuver under water like a submarine. In the second scheme, the plan configuration is altered to cater for the present destroyer class vessels. These two schemes are shown respectively in Drawings 1 to 7 inclusive.

The structural configuration of the pier is generally as shown in the original concept. The elevation of the main deck is set at 17' above water. The draft of the finger piers, which are made up basically of buoyancy chambers, is 13' on the average, while the draft of the main spine pier is 27'. As mentioned previously, the pier is to be made up of several modules. For each unit to be self-floating, the finger and spine piers will have to be enclosed around the periphery to keep the water out. Connection details between the modules when they are maneuvered into position are shown in Drawing 11. For Scheme 2 where the pier is shown to service the present ships, the finger piers are set at 330' clear apart to meet the requirements of the design manual.

Service lines are led to each finger pier from the second deck of the spine pier, either through a servicing tower as shown in the original concept, or by pipe troughs under the finger piers, as shown in Drawing 11. If the servicing tower to the stern or the bow of a ship is envisaged, the utility inlets of the ship will have to be located accordingly.

The pier is anchored by two pylons, one at each end of the spine. As envisaged, the pylon consists of a hollow circular shaft or shaft of other cross-sections, which is inserted into a base structure on the floor basin. The advantage of this is the simplicity of the process of installing and removing anchors, thereby enhancing the relocatability of the pier. The procedure of anchoring or deanchoring consists of no more than inserting into or removing the vertical shaft from the base structure. See Drawing 10 for possible details.

The pier main deck is connected to the shore by one 20' wide ramp and the second deck by two 10' wide ramps, one for ingress and the other for egress. These ramps are connected to a pinned joint at the shore end, and a sliding joint at the pier that are designed to accommodate relative vertical and horizontal movements.

The availability of the second deck will provide the necessary space required to make the pier as self-sufficient as possible. There will be ample space for storage, living accommodations for the maintenance and control crew, etc.

It is also possible to convert the spine deck into a runway for Navy planes, similar to the conversion of the finger pier in the expeditionary pier concept into runways. In this case, the pylon must not protrude above the deck level, and service lines must be kept below the main deck.

5. VALIDITY OF DESIGN

Based on the design considerations listed above, the preliminary design of the pier was carried out and the calculations enclosed as Appendix A at the end of this report. As for the expeditionary pier, this design was checked to ensure that the structural system as quantified in the drawings, will be adequate to withstand assumed construction, towing and operating conditions and that its flotation and naval architectural characteristics are satisfactory. For this pier the major design problems are centered around:

1. The connection between the modules;
2. The anchoring system;
3. The supply of services for a high concentration of ships that the pier can accommodate; and
4. The shore-pier interface.

The rest of the design is seen to be within the state of the art.

6. ANALYTICAL AND DESIGN FEASIBILITY

The feasibility of design of this pier is similar to that described for the expeditionary pier in Report No. 1. Deficiencies in the state-of-the-art technology are nevertheless present, particularly in relation to the major problem areas as pointed out in the previous section. This will be commented upon in the following.

6.1 Connection Between Modules

Connections between prefabricated modules of a large floating structure are within the state of the art. The difficulty in our case is with the size and lightship draft of the modules involved. Our design has called for module connection between spine and spine of similar draft, and finger to spine modules of different drafts. This last connection requires ballasting of the finger module to achieve the same draft as the spine pier. The two modules assembled are first of all maneuvered into position and secured in the relative position by a set of temporary ties on the main deck. The two modules to be connected are then pulled together by tightening the adjustable ties at the sides of the connecting face as shown. This will provide sufficient force for the two connecting faces to seal off the trench between the two adjacent bulkheads along the joint. Water is pumped out of the trench to permit prestressing couplings and other reinforcing steel to be connected or spliced. The trench is then concreted, and when the concrete has acquired the necessary strength, the connecting tendons are stressed.

6.2 The Pylon Anchoring System

The purpose of the pylon is to restrain the pier against horizontal movement while permitting it to move vertically with the change in water level. It consists of a larger-diameter hollow vertical shaft standing freely in a sleeve that forms part of a piled base structure. It is important to keep the floating pier aligned with the pylon base structure when the pylon is being positioned. The tentative anchoring system is shown in Drawing 10.

6.3 Utility Lines

In view of the number of ships that could be serviced by this pier and of the need for the pier to be as self-sufficient as possible, it is envisioned that the pier will be equipped to generate most if not all of the services, such as steam, power, sewage disposal, and potable and fire-fighting water supplies. Because of the size of the supply system, the flexible connection of the utility lines between the pier and the shore would be an area of major design effort, as this connection must accommodate the full range of vertical movements due to water action, and limited horizontal displacements caused by environmental loads. Refer to Dwgs. 8 & 9.

7. MATERIAL FEASIBILITY

Normal weight concrete has been used in this design as the prime structural construction material. No difficulty is foreseen in the use of this material, which has been proven in marine applications. Permeability and abrasion of the pier must be kept to a minimum in order to ensure long-term durability. The standard way to watertightness is to produce good concrete, provide proper curing, and coat the exterior face of the construction joints with sealant. Lightweight concrete could be used to advantage for floating structures. Lightweight concrete can now be made with high working strength (7,000 psi) and resistance against abrasion. Concrete in the sea inhibits marine growth due to its alkalinity, although growth does take place. Where this problem exists, the surface could be coated with marine growth inhibitors commonly used for offshore work. Other materials that can be used include high strength steel tendons and wires, mild steel and wire mesh, all of which are readily available today.

The materials that may require some research will be those connected with the construction of the pylon anchoring system and materials used in the connection of the utility lines across the shore-pier interface. The pylon shaft requires a large cross-section and can be exceedingly heavy. Its movement will require the use of very heavy cranes, which may not be easily available at a particular site. The design tentatively calls for 13' outside diameter normal weight concrete pylons weighing 300 tons each. As an unconventional alternative, high strength polymer concrete with three times the conventional strength may be used to reduce weight.

8. CONSTRUCTIONAL FEASIBILITY

The comments that were made for the expeditionary pier are mostly equally applicable in the case of the marina pier. Generally it is envisioned that the pier modules will be built by the flood basin method, as described for the expeditionary pier. The same flood basin method may be used to construct the base structure of the pylon anchors and the pylons.

9. ALTERNATIVE ANCHORING SYSTEM

It will be possible to anchor the marina pier by the conventional anchor and mooring line methods. However, the anchoring system will be extensive, requiring perhaps 16 or 18 anchoring points, making the installation and removal of the anchors a lengthy process. Another difficulty is the possible excessive excursion of the pier while on station, and the problem it poses to its connection with shore.

It may also be possible for this marina pier to incorporate the stiff-legged anchoring system as described for the expeditionary pier.

10. COST CONSIDERATIONS

Based on the concept presented herein and the unit prices, as listed in the following, the cost of the pier may be estimated as follows:

For hard rock concrete the overall unit cost is taken at $900/C.Y. of concrete. This order-of-magnitude cost includes concrete, steel, ramps, construction, tow and connection, fenders, plus an additional allowance of 10% for miscellaneous items. The cost of the pier structure then works out to be $85 million for approximately 95,000 C.Y. of concrete.

The cost of the pier structure, if constructed of lightweight concrete assumed at $1,000/C.Y. of concrete, will amount to $95 million. On a unit cost basis, this works out to be $164 per sq. ft. for a total of 578,000 sq. ft. of deck span including the second deck for Scheme 2.

11. CONCLUDING REMARKS

The development of this concept had produced a number of novel features which may have important impact on the development of floating piers in general. The most significant ones are the pylon anchoring system, and the unprecedented magnitude of vessel size.

The technological feasibility of the marina pier has been demonstrated in this brief exercise. The initial cost is high compared to the conventional pier. It does offer important advantages, as described in this report if conditions exist to justify its construction.

Appendix A ; Scheme 2

Geometry

Size : Pylon : 13'ϕ_o , 11'ϕ_{in} x ~55'
Spine ~1000' x 160'
Fingers ~ 540' x 80' each

Hardrock : Dead weight : 203360 T
Lightship draft : 17.3'
Live load draft : 10.5' @ Full load
Total 27.8'

Light weight : Dead weight : 179000 T
Lightship draft : 14.0'
Live load draft : 10.5' @ Full load
Total 24.5'

Center of Gravity and Buoyancy located at the Geometric center of the pier (x, y plane).

Pier Period$_{x-x}$ = 9.3 sec (Hardrock concrete)

Loading

Sea State 2 : Max. Wave Height : 3'
Wind Velocity : 50 mph
Current : 3 knots

Materials

Hardrock Concrete @ 150 pcf or
Lightweight Concrete @ 120 pcf
F'c = 7000 psi For Hardrock and Lightweight concrete
Prestressing Steel : Uncoated Seven wire strand
ASTM A416, 5/8"ϕ, Fpu = 270 psi
Reinforcing Steel : ASTM A615 Grade 60

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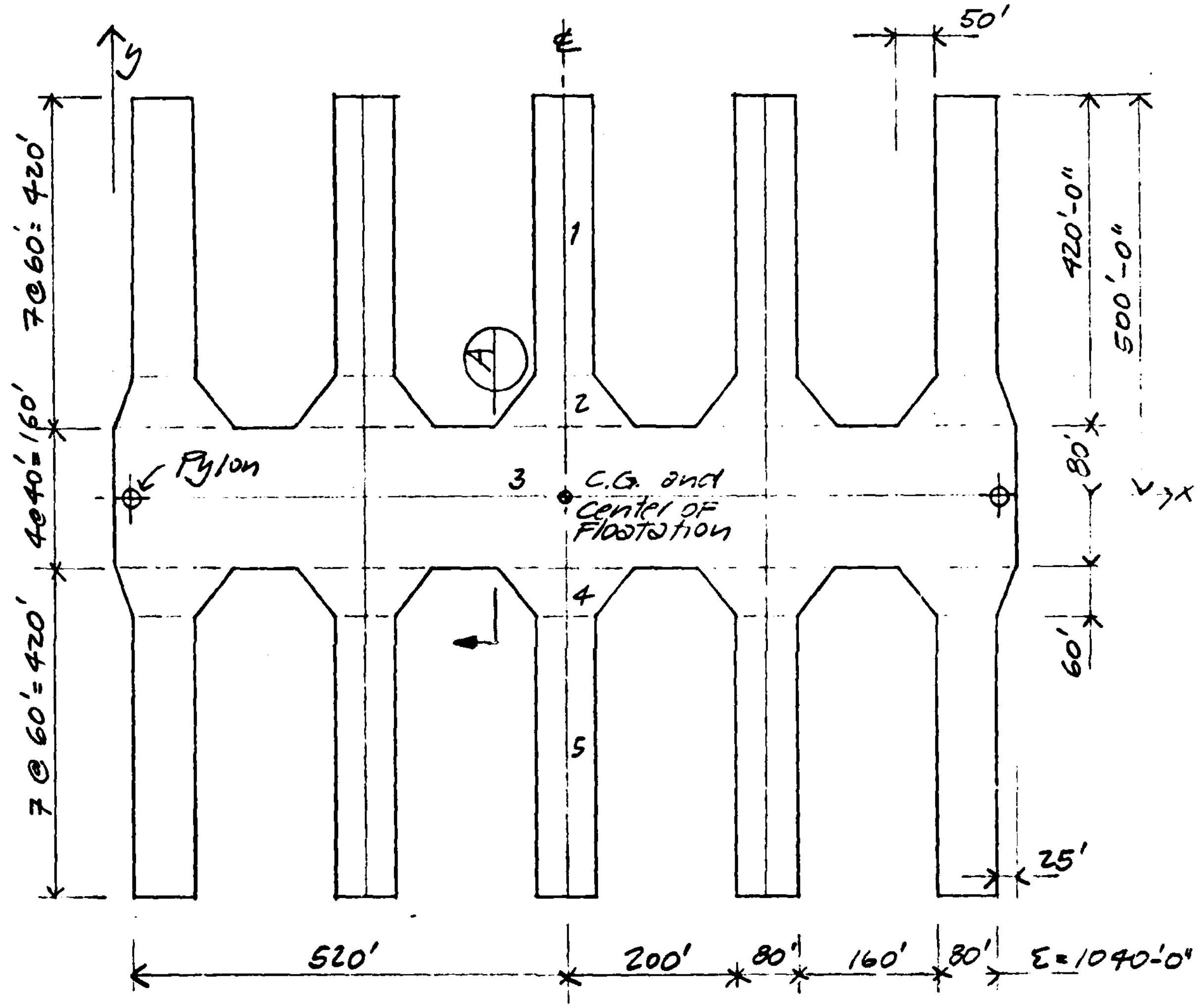
STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca. 94133.

PROJECT:	Navy Pier
ITEM:	Floating Marine Pier, Sch. 1
DESIGN:	Surface Area & Floatation
DATE:	11 93 KM
SHEET	A-1.1
OF	
REVISION	



	Area$^{\phi}$	$y^{(1)}$	$Ay^{(F^3)}$	$Ay^{2\,(F^4)}$	$I_{yo}^{(F^4)}$
1	144000	820	118×10^6	9.68×10^{10}	1.55×10^9
2	42000	614	25.8×10^6	1.58×10^{10}	1.18×10^7
3	174400	500	87.2×10^6	4.36×10^{10}	3.72×10^8
4	42000	386	16.2×10^6	6.25×10^9	1.18×10^7
5	144000	180	25.9×10^6	4.67×10^9	1.55×10^9
	546400		293×10^6	170.6×10^9	

$$I_{xx} = 170.6\times10^9 - 546400\times500^2$$
$$= 3.4\times10^{10}$$

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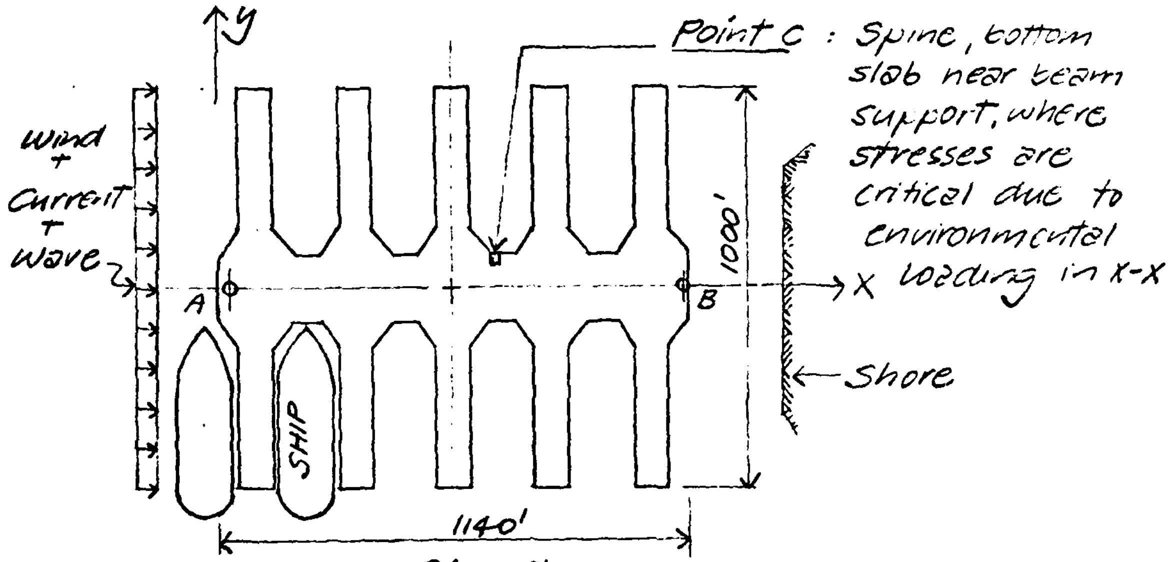
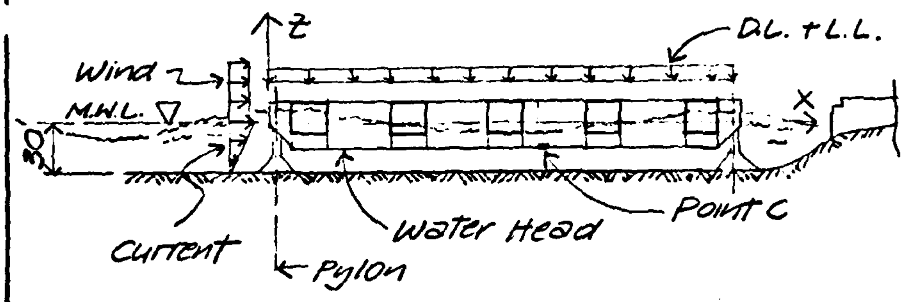
PROJECT: Navy Piers	SHEET: A-1.2
ITEM: Floating Marina Pier	OF
DESIGN: Loading, Scheme 1	REVISION:
DATE: 1/83 KM	



Pier Plan



Loads in the Structure :

a) Permanent : - Dead Load concrete + pavement
- Water head due to Dead Load

b) Live Load : - 400 psf @ top deck , 150 psf @ middle deck
- 200 psf @ bottom deck
- water head due Live Load, mooring of vessels

c) Deformation : - Prestressing
- ΔTemperature , Shrinkage , creep (ignore now)

d) Environmental : - Wave Load : 3' high wave, L = 1140'
(Sea State 2) - Current @ 3 Knots @ water surface.

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PROJECT: Navy Pier	SHEET: A-13
ITEM: Multiple Marina Pier, Sch. 1	OF
DESIGN: ABS Rules For Wave Loads	REVISION:
DATE: 11'83 RM	

Use ABS Rules ; Environmental Loads For Sea State 2

Section 6.3.2 Total Bending Moment about y-y axis

$Mt_{yy} = Msw_{yy} + Mw_{yy}$

Msw_{yy} = Still Water Bending Moment
= 0 Assuming : Uniform D.L & L.L. distribution

Mw_{yy} = Maximum Wave Induced Bending Moment
$= C_2 L^2 B He K_b$

$K_b = 1.0$ For $C_b > 0.80$

$C_b = 1.0$ assumed

$C_2 = [6.53\, C_b + 0.57] \times 10^{-4}$
$= [6.53 \times 1.0 + 0.57] \times 10^{-4}$
$= 7.1 \times 10^{-4}$

$L = 1040' + 100' = 1140'$ (x-x axis)

$\overline{B} = (1000' \times 80' + 220' \times 50' + 160' \times 60')/190'$
$= 529'$ (weighted average)

$He = 26.75'$

$Mw_{yy} = 7.1 \times 10^{-4} (1140)^2 \times 529' \times 26.75' \times 1.0 \times 2.24$
$= 29.2 \times 10^6$ K-F

Determine Allowable Moment that can be resisted by pier @ the 160' wide section:

Sea state ⟶ Max. Wave Height = 3'

$$Mmax_{yy} = \frac{H}{He} \times Mw_{yy} \quad @ \text{ Midship}$$
$$= \frac{3}{26.75} \times 29.2 \times 10^6$$
$$= 3.28 \times 10^6 \text{ K-F}$$

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PROJECT:	Navy Pier
ITEM:	Floating Marina Pier, Sch. 1
DESIGN:	Environmental Loads 1-X
DATE:	11 83 KM
SHEET:	A-17
OF	
REVISION:	

ABS Wave Bending Moment and Shear Distribution

$M_{max\,yy} = 3.28 \times 10^6$ K-F $L_x = 1140'$

Stations @ 114-0"

Station	Bending Moment (Wave Height = 3')			
0	0	=	0	K-F
2	$0.10 \times 3.28 \times 10^6$	=	330000	
4	$0.35 \times 3.28 \times 10^6$	=	1150000	
6	$0.68 \times 3.28 \times 10^6$	=	2230000	
8	$0.95 \times 3.28 \times 10^6$	=	3120000	
10	$1.00 \times 3.28 \times 10^6$	=	3280000	
12	$0.94 \times 3.28 \times 10^6$	=	3080000	
14	$0.74 \times 3.28 \times 10^6$	=	2430000	
16	$0.43 \times 3.28 \times 10^6$	=	1410000	
18	$0.13 \times 3.28 \times 10^6$	=	430000	
20	0	=	0	

Shear Force

0	0	=	0. K
2	$(330000/114) \times 2$	=	5789
4	$[(1150000 - 330000 - 5789 \times 114)2]/114 + 5789$	=	8597
5	$0.45 \times 1140 + 8597$	=	9110
6		=	8597
8		=	5789
10	0	=	0
12		=	5789
14		=	8597
15		=	9110
16		=	8597
18		=	5789
20	0	=	0

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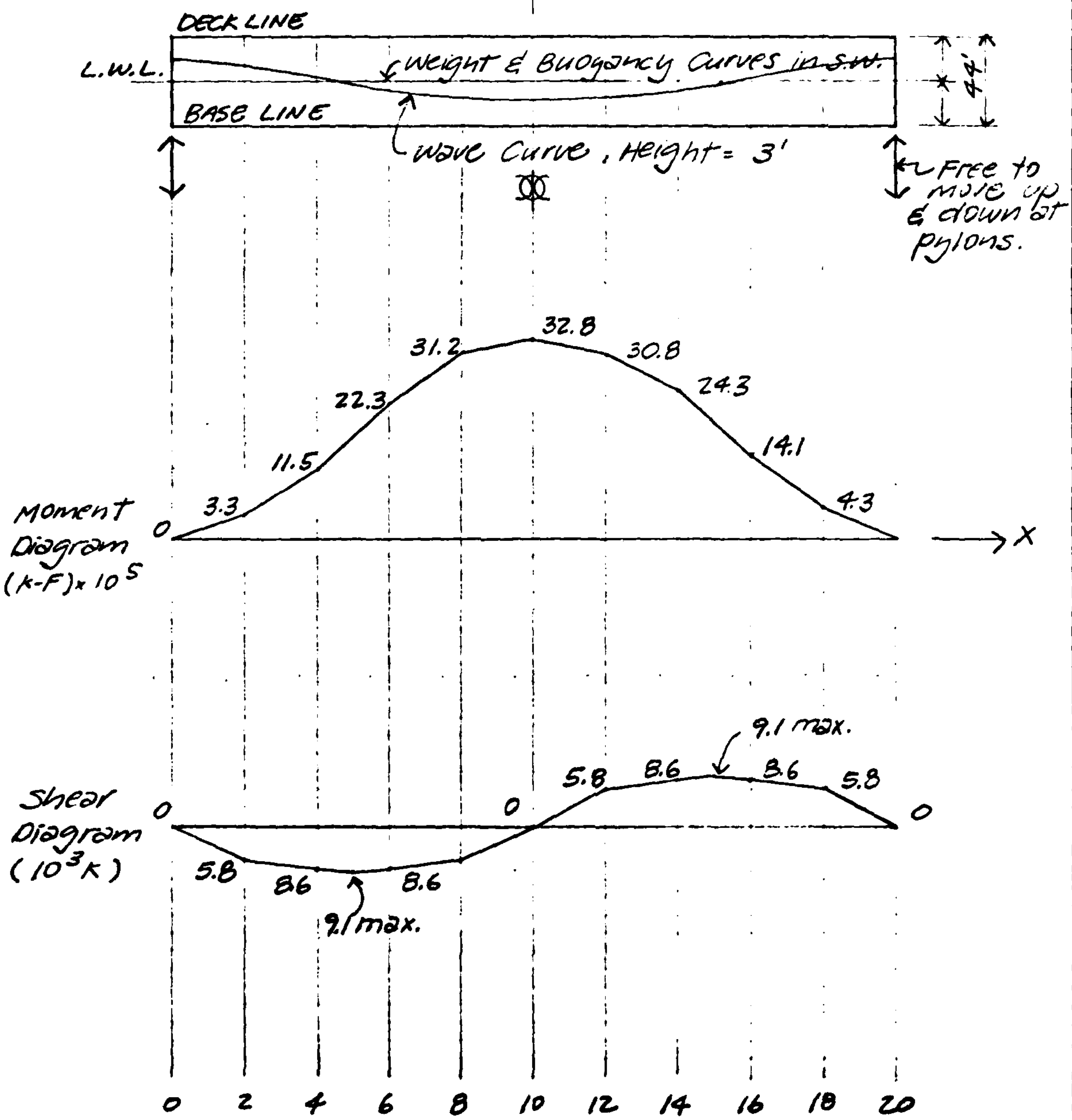
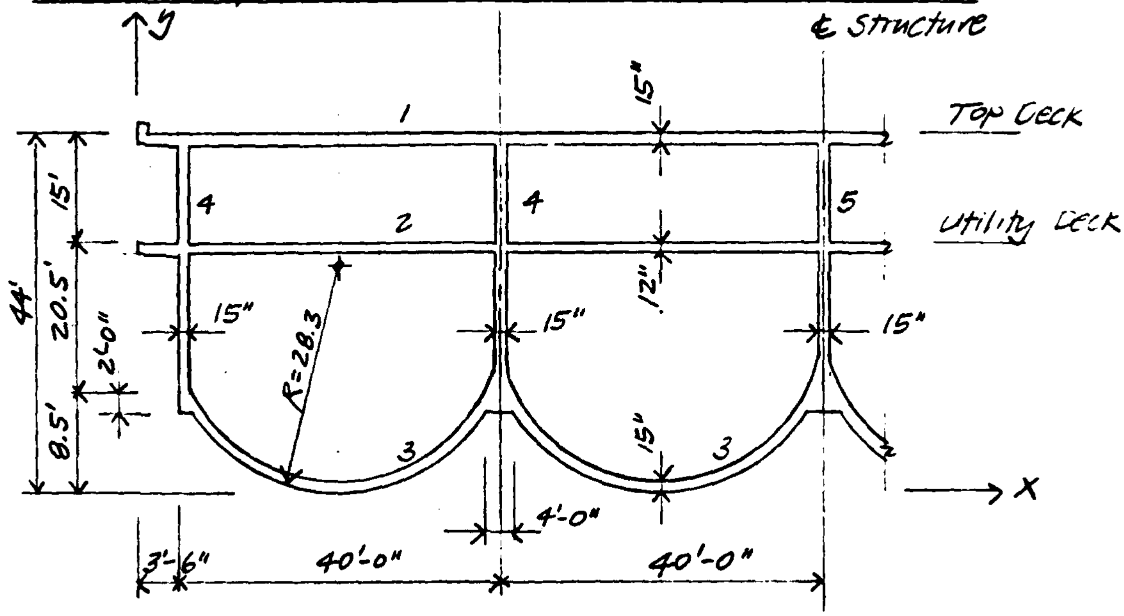
PROJECT: Navy Pier	SHEET: A-1.5
ITEM: Multiple Marina Pier	OF
DESIGN: Environmental Loads – Waves	REVISION:
DATE: 11 83 KM	

Wave Moments and Shear Envelopes (y-y axis)
Scheme 1



T.Y. LIN INTERNATIONAL STRUCTURAL ENGINEERING 315 Bay St., San Francisco, Ca. 94133	PROJECT: Navy Piers ITEM: Floating Marina Pier, Sch. 1 DESIGN: Section Properties DATE: 11 83 KM	SHEET: A-1.5 OF REVISION:

Section Properties - Trial Section II, Scheme 1



CROSS-SECTION @ (A)
(I)

	A^{F^2}	y^F	Ay^{F^3}	Ay^{F^4}	I_{ox}	y_t / y_b	S_t / S_b
1	101.1	43.4	4384	190158	13		
2	80.9	28.5	2306	65711	7		
3	108.7	2.75	299	822	317		
4	46.9	25.3	1186	30005	5496		
5	23.5	25.3	595	15054	2748		
	361.1	24.3	8770	301750	8581	19.7 / 24.3	

$I_{xx} = [(301750 + 8581) - 361.1 \times 24.3^2] \times 2$
$= 194210\ F^4$

$S_b = 194210 / 19.7 = 9858\ F^3 < 12974\ F^3$

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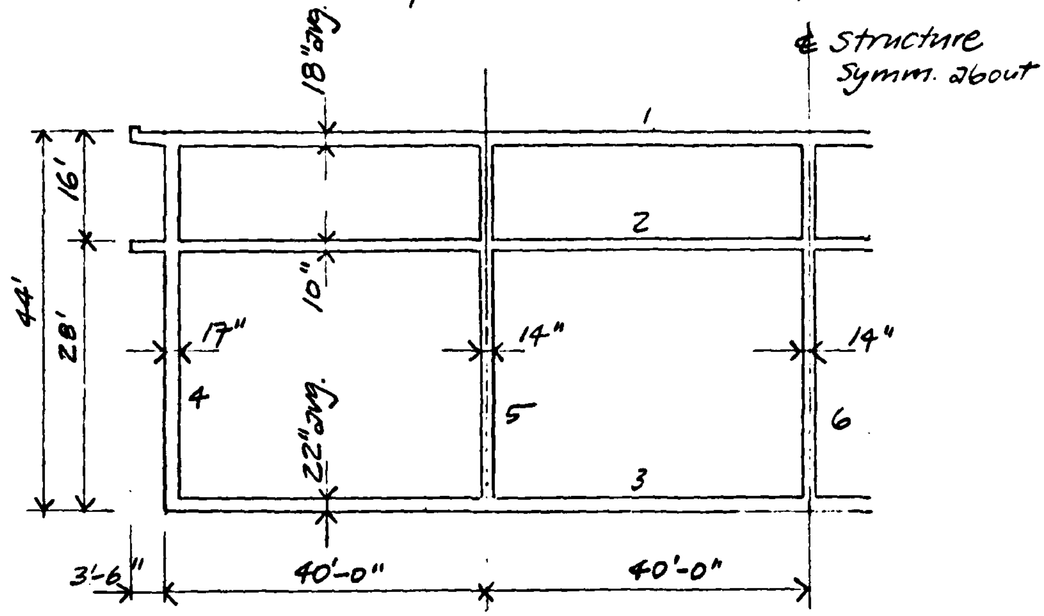
PROJECT: Navy Piers
ITEM: Floating M. Pier, Scheme 1
DESIGN: Section Properties @ (A)
DATE: 11/83 KM
SHEET: A-17
OF
REVISION:

Critical Section Properties near Midship, Trial Section II



Section @ (A)
(II)

	A F^2	y F	Ay F^3	Ay^2	I_{ox}	y_t / y_b	S_t / S_b
1	120.4	43.3	5207	225216	23		
2	66.8	27.6	1843	50855	4		
3	140.9	0.92	129	118	39		
4	62.3	22	1371	30169	10056		
5	51	22	1122	26684	8282		
6	25.7	22	565	12439	4141		
	467.1	21.9	10237	345481	22545	22.1	13002
						21.9	13121

$I_{ox} = [(345481 + 22545) - 467.1 \times 21.9^2] \times 2$
$= 287342 \ F^4$

$S_b = 287342 / 21.9$
$= 13121 \ F^3 > S_{req}$ ✓

$S_t = 287342 / 22.1$
$= 13002 \ F^3 > S_{req}$ ✓

Minimum Allowable Section Modulus at Section (A)

$$M_{max} = \left[32.8 - \left(\frac{32.8-31.2}{114}\right) \times 90\right] \times 10^5$$
@ (A)
$$= 3153648^{k\text{-}F}$$

$\sigma_{b\,max}$ = Maximum Allowable Bending Stresses:
$= 0.45\, F'c/2$ $\qquad F'c = 7500$ psi
$= 1688$ psi

Minimum $S_{t,b} = M_{max}/\sigma_{b\,max}$
@ Section (A)
$= 3153648/(1688 \times 144/1000)$
$= 12974^{F^3}$

This is the critical section of Pier for wave loading parallel to x-axis because of the high moment and the small width at this location.

Section Modulus provided at this Section: See pg.

$S_t = S_b = 13000^{F^3} > 12974^{F^3}$
$\therefore$ Satisfactory in bending M_{yy}

Maximum Wave Height Allowable for $S = 20041^{F^3}$:

$$H = \frac{M_{max_y} \times He}{M_{wave_y}(\;)}$$

$M_{max_{yy}} = (S_{t,b})\,\sigma_{b\,max}$
$= 13002 \times 1688 \times 144/1000$
$= 3.16 \times 10^6$ k-F

$$H = \frac{3.16 \times 10^6}{3.15 \times 10^6} \times 3.0'$$

$= 3.01'$ wave

Comparison of Cross Sections @ Midship : Scheme 1

A rectangular cross section is preferable compared to a semicircular one. The biggest loading here comes from wave action in the long direction of the pier. The pier here acts like a flexible beam which is stressed to its maximum @ the top and bottom slabs. These stresses account for ~ 75% of the total stress at the bottom slab. Therefore the higher the Moment of Inertia of the Section the more effective it is.

Since the operational depth of water is limited to ~ 40' the pier draft must also be limited to ~ 30' max. Therefore the section height is controlled by this parameter.

A circular section, like Section I, is quite effective for high hydrostatic pressures and torsion but not so for forces that causes bending stresses in one direction (wave action) like in our case. As hull depth is shallow, hydrostatic pressure is small and therefore a circular section is not apropiate.

Forming of a circular shape become more expensive and module connections more difficult than rectangular shapes.

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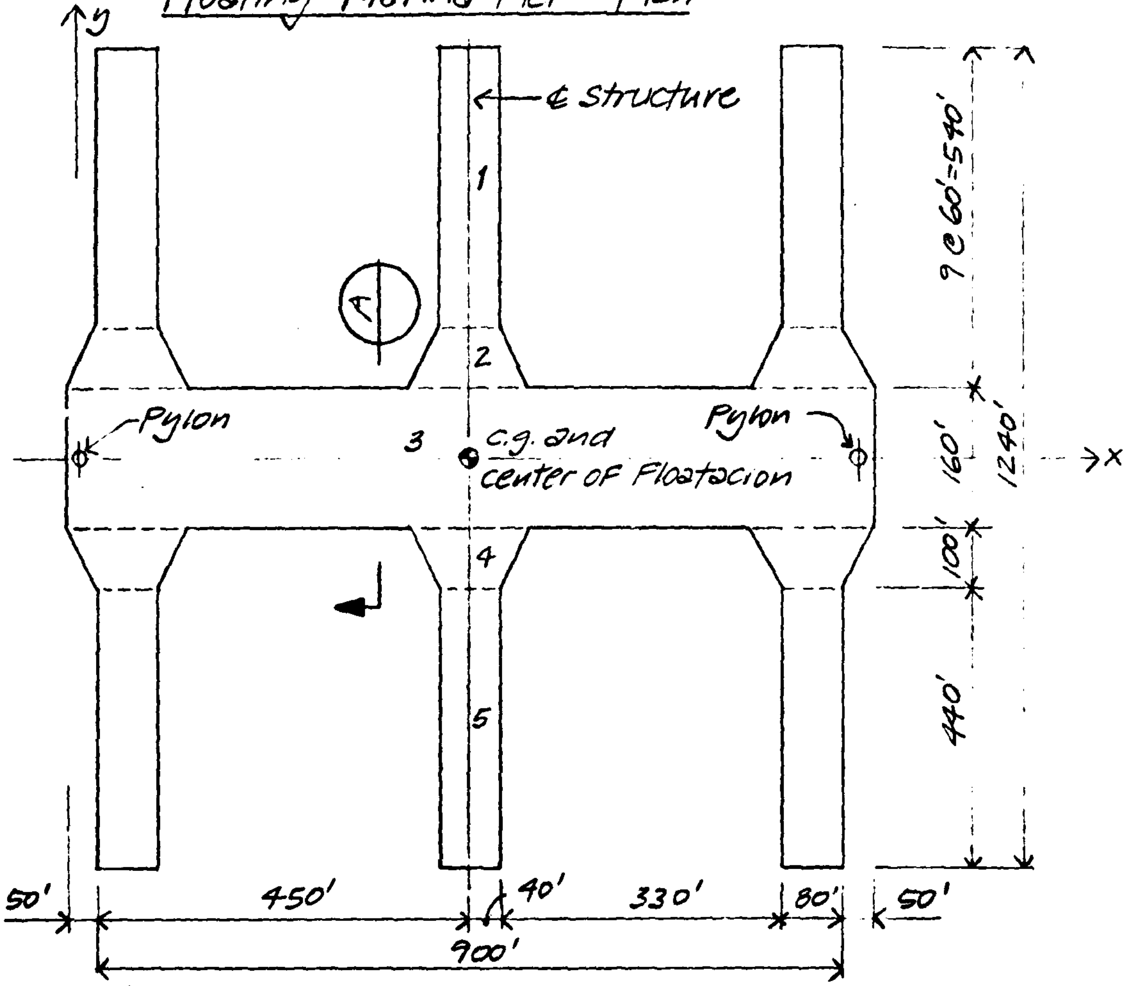
Floating Marina Pier Plan



	Area (F²)	y (F)	Ay (F³)	Ay² (F⁴)	I_{y_0} (F⁴)
1	105600	1020	1.08×10^{8}	1.1×10^{11}	1.7×10^{9}
2	31500	746	2.35×10^{7}	1.75×10^{10}	8.6×10^{6}
3	144000	620	8.9×10^{7}	5.54×10^{10}	3.1×10^{8}
4	31500	494	1.5×10^{7}	7.7×10^{9}	8.6×10^{6}
5	105600	220	2.3×10^{7}	5.1×10^{9}	1.7×10^{9}
	418200	620'	2.59×10^{8}	1.96×10^{11}	0.03×10^{11}

$$I_{xx} = 1.99\times10^{11} - 418200\times\overline{620}^{2}$$
$$= 3.8\times10^{10}\ F^4$$

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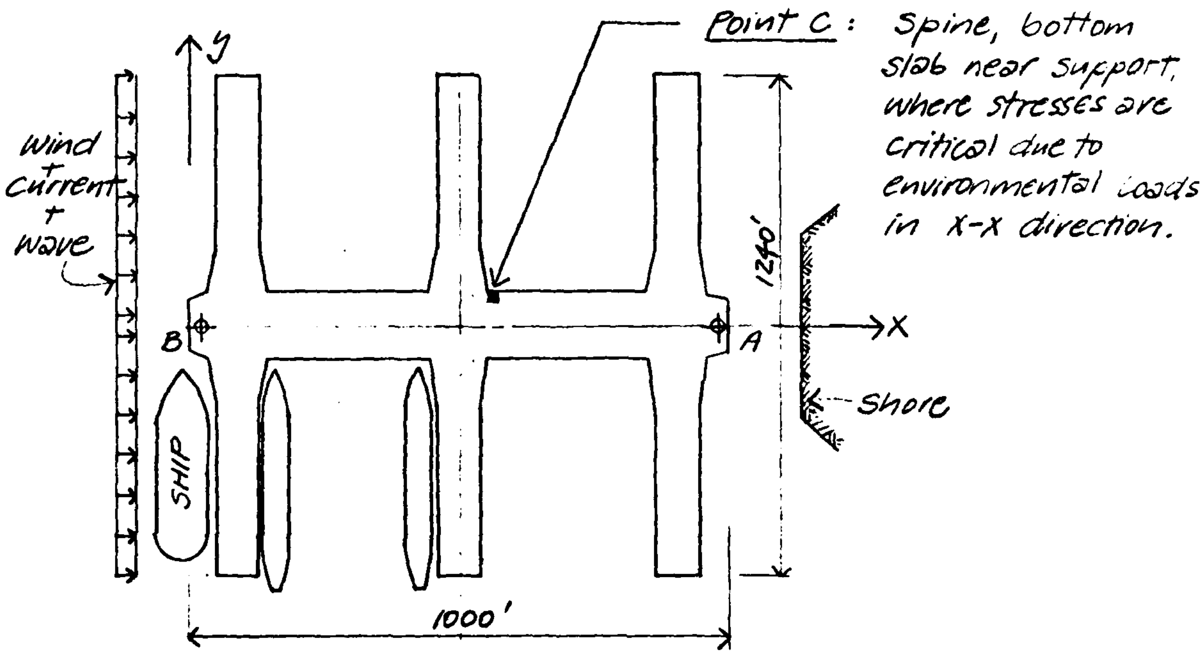
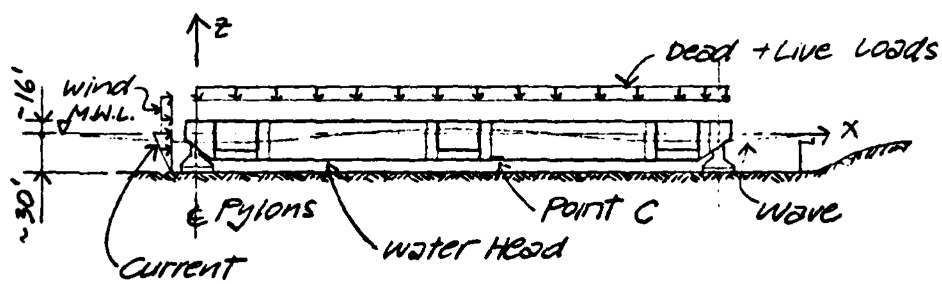


Pier Plan



Loads in Structure :

a) Permanent : Dead Load : Concrete, mechanical + pavement. Water Head due to Dead Load.

b) Live Load : 400 psf @ top deck , 150 psf @ middle deck 200 psf @ lower deck
Water Head due to Live Load, mooring of vessels.

c) Deformation : Prestressing , Δ Temperature + Shrinkage + Creep

d Environmental : Wave Load : 3' high wave , L = 1000'
(Sea State 2) Current @ 3 Knots @ Water Surface

Structural System :

The system consists of a Floating concrete-box right angle, multiple berth pier kept in place by 2 - Anchor pylons, one at each end of the pier.
The concrete box (see Dwg. 6) carries all external vertical loads: D.L., L.L., water Head, Wave Sagging and Hogging, through local and global axial, shear and bending as the pier is free to move vertically at the pylons.

All horizontal forces due to wind, Berthing and Current will be transfered to the pylons through axial, and shear deformations in the concrete-box structure. These external loads will be eventually carried to the soil by piles set in the pylon foundation.

State of Stresses @ a Critical Point

Point C : Critical in vertical loading as global bending stresses (primary stresses) due to wave hogging are highest.

Loading	Stresses at Point C	
	Local	Global
D.L.	$Bending_{yy}$	——
L.L.	"	——
*1 Buoyancy	"	$Axial_{x,y}$
Wave S. & H.	——	$Bending_{yy}$
*2 Wind	——	$Axial_{x,y}$
*2 Current	——	$Axial_{x,y}$
Prestressing	$Bending_{xx,yy}$	$Axial_{x,y}$

*1 assume no internal pressure due to fluids in tanks
*2 assume same number of ships nested at either side of spine pier (balance horizontal reaction)

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	DATE: 11/83 KM	

Cont' Loads in the Structure:

d) Environmental : - Wind @ 15 Knots

e) Accidental : - Collision (ignore)
- Explosion etc (ignore)

Load Combination: (NAVFAC DM-26)

Group	Loading	% Unit All. Stress
a	$D+L+I+E+B+W_a+Be+P$	100%
d	Group a. + 30% $W + W_s$ + Wave $+ C + S_s + S + T$	140%

D : Dead Load
L : Live Load
I : Impact (ignore)
E : Earth Pressure (non existing)
B : Buoyancy
W_a : Water Pressure
Be : Berthing Load
P : Prestressing
W : Wind on Structure
Ws : Wind on Ships
wave : Hogging or Sagging, Surge, sway
C : Current
S_s : Surge, Sway (ignore as have Fixed moorings)
S : Shrinkage (ignore)
T : Temperature (ignore)

ABS Wave Moments and Shears

Section 6.3.2 (Total Bending Moment about y-y axis due
Sea State 2 to waves in the x direction)

$Mt_{xx} = Msw_{xx} + Mw_{xx}$

Ms_{xx} = Still Water Bending Moment
= 0 Assuming uniform D.L. & L.L. distribution

Mw_{xx} = Maximum Wave Induced Bending Moment
$= C_2 L^2 B He K_b$

$K_b = 1.0$ For $C_b > 0.80$
$C_b = 1.0$ assumed
$C_2 = [6.53\, C_b + 0.57] \times 10^{-4}$
$= [6.53 \times 1.0 + 0.57] \times 10^{-4}$
$= 7.1 \times 10^{-4}$
$L = 1000'$ (x-x axis)
$B = 418200^{\#}/900' = 465'$ (average)
$He = [4.5 \times 1000' - 0.00216 \times 1000^2 + 335] \times 10^{-2}$ $720 < L \leq 1000'$
$= 26.75'$

$Mw_{xx} = 7.1 \times 10^{-4} (1000)^2 \times 465' \times 26.75' \times 1.0 \times 2.24$
$= 19.8 \times 10^6$ K-F

Determine Allowable Moment that must be resisted by pier @ the 160' wide section :

Sea State 2 ⟶ Max. Wave Height = 3'

$$Mmax_{xx} = \frac{H}{He} \times Mw_{xx} \quad @ \text{ Midship}$$
$$= \frac{3'}{26.75'} \times 19.8 \times 10^6$$
$$= 2.22 \times 10^6 \text{ K-F}$$

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DATE:	11/83 KM
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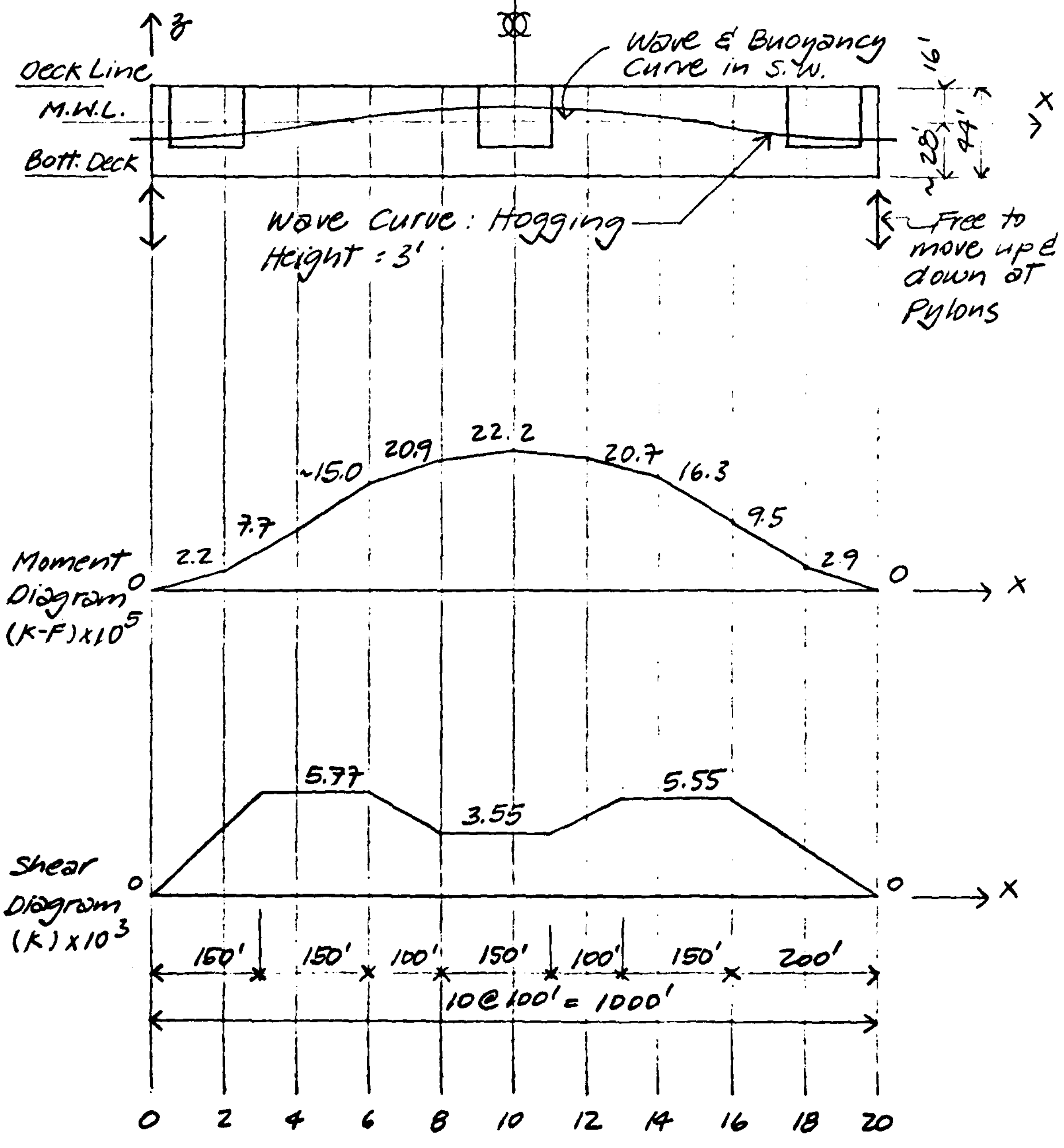
REVISION:	

Wave Moments and Shear Envelopes (x-x direction)



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ABS Wave Bending Moment and Shear Distribution, x-x

$Mmax_{yy} = 2.22 \times 10^6$ K-F L = 1000'

Stations at 100'-0" along x axis

Station	Bending Moment (Wave Height = 3')			
0	0	=	0	K-F
2	$0.10 \times 2.22 \times 10^6$	=	220000	
4	$0.35 \times 2.22 \times 10^6$	=	770000	
6	$0.68 \times 2.22 \times 10^6$	=	1496000	
8	$0.95 \times 2.22 \times 10^6$	=	2090000	
10	$1.00 \times 2.22 \times 10^6$	=	2220000	← Mmax. at midship
12	$0.94 \times 2.22 \times 10^6$	=	2070000	
14	$0.74 \times 2.22 \times 10^6$	=	1628000	
16	$0.43 \times 2.22 \times 10^6$	=	954600	
18	$0.13 \times 2.22 \times 10^6$	=	288600	
20	0	=	0	

Shear Distribution

$Fmax_{xx} = Fs_{yy} + Fw_{yy}$

Fs_{xx} = Still Water Shear
= 0

$Fw_{xx} = K\,Mw_{xx}/L$

Mw_{xx} = Max. wave moment X.
= 2220000 K-F

L = 1000'

K = 0 at ends
= 2.6, $0.15L < y < 0.30L$
= 1.6, $0.4L < y < 0.55L$
= 2.5, $0.65L < y < 0.8L$

K	Fw_{xx} (k)
0	0
2.6	5772
1.6	3552
2.5	5550

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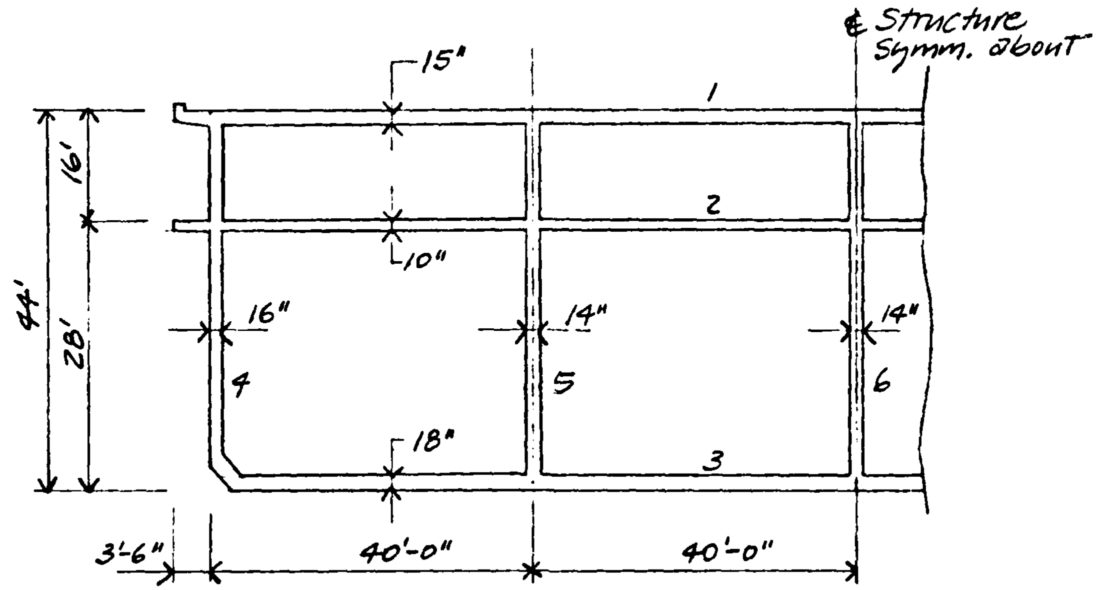
PROJECT: Navy Piers
ITEM: Floating Marina Pier, Sch. 2
DESIGN: Section Properties @ A
DATE: 11/83 KM

SHEET: A-2.3
OF
REVISION:

Critical Section Properties near Midship :



Section @ A

	A (ϕ)	y (Ft)	Ay (F^3)	Ay^2 (F^4)	Iox (F^4)	yt / yb	St / Sb
1	100.5	43.4	4363	189337	13		
2	66.8	27.6	1843	50855	4		
3	115.4	0.75	86	64	22		
4	58.7	22	1291	28395	9465		
5	51	22	1129	24838	8281		
6	25.7	22	565	12423	4141		
	418.1	22.2	9277	305914	21926	21.8	10971
						22.2	11173

$Iox = [(305914 + 21926 - 418.1 \times 22.2^2] \times 2$
$= 243563 \; F^4$

$St = 243563/21.8$
$= 10971 \; F^3 > S_{req} = 9418 \; F^3$ ✓

$Sb = 243563/22.2$
$= 11173 \; F^3 > S_{req} = 9418 \; F^3$ ✓

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PROJECT: Navy Pier
ITEM: Floating Marina Pier, Sch. 2
DESIGN: Environmental Loads, X-X
DATE: 7/1 83 RM

SHEET: A-29
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Minimum Allowable Section Modulus @ Section (A)

$M_{max@A} = [2.22 - (2.22-2.09)/100' \times 65'] \times 10^6$

$= 2136000^{K-F}$

σ_{bmax} = Maximum Allowable Bending Stresses :

$= 0.45 f'c/2 \qquad f'c = 7000^{psi}$

$= 1575$ psi

Minimum $S_{t,b} = M_{max}/\sigma_{bmax}$
at point (A)

$= 2136000/(1575 \times 144/1000)$

$= 9418\ Ft^3$

This is the critical section of the pier for wave loading parallel to the X-axis because of the high moment and the narrow width at this location.

Section Modulus provided at this Section : See pg.

$S_t = 11173^{F^3}$

$S_b = 10971^{F^3} \quad > 9418^{F^3}$

$\therefore$ satisfactory in bending

Maximum Wave Height Allowable For $S_b = 10971^{F^3}$:

$$H = \frac{M_{max\,xx}}{M_{wave\,xx}(3')} \times He$$

$M_{max.} = S_b \times \sigma_{bmax.}$

$= 10971 \times 1575 \times 144/1000$

$= 2.49 \times 10^6$ K-F

$$H = \frac{2.49 \times 10^6}{2.14 \times 10^6} \times 3'$$

$= 3.5'$ wave

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PROJECT: Navy Piers	SHEET: A-2.10
ITEM: Floating Marina Pier, Sch. 2	OF
DESIGN: Environmental Loads: Wind	REVISION:
DATE: 11/83 KM	

<u>Wind Loading in x-x direction</u>

Assume : a) Sea state 2, Wind Velocity of 50 MPH in x-x direction, i.e, @ 90° to the Ships

b) All ships nested (12) in the Pier.

c) Ships nested in Leeward side of every Finger will take 50% of wind loads on ships at the windward side of the Pier.

12, DD-963 nested at Pier : 6, DD-963, 100% exposure
6, DD-963, 50% exposure

<u>From DM 26</u> : $F_W = C_{yw} \times \frac{1}{2} \rho_W V_W^2 A_S$

Wind @ 90° to ship, $C_{yw} = 1.0$

$\rho_W = 0.00237 \ \frac{\#-sec^2}{F^4}$ @ 68°F

V_W = Velocity, F/sec @ 40' Above waterline
= 73 F/sec

<u>For DD-963</u>

DM 26.6 Gives data on smaller DD's

DD-692	L = 377'	h = 22'	As = 10200 ☐
DD-931	L = 418'	h = 31'	As = 13000 ☐

Extrapolating

<u>DD-963</u>	L = 564'	h = 39.6'	As = 22600 ☐

At 100% exposure:

$(F_W)_{DD} = 1.0 \times (\frac{1}{2})(0.00237)(73)^2(22600)$
$= 144 \ ^K/\text{ship (windward)}$ ⟵

At 50% exposure :

$(\frac{1}{2})(F_W)_{DD} = 144/2 = 72 \ ^K/\text{Leeward ship}$ ⟵

<u>For Floating Pier</u> : $A_S = 16' \times 200' = 3200$ ☐

$(F_W)_{F.P.} = 144 \times 3200/22600 = 20^K$ ⟵

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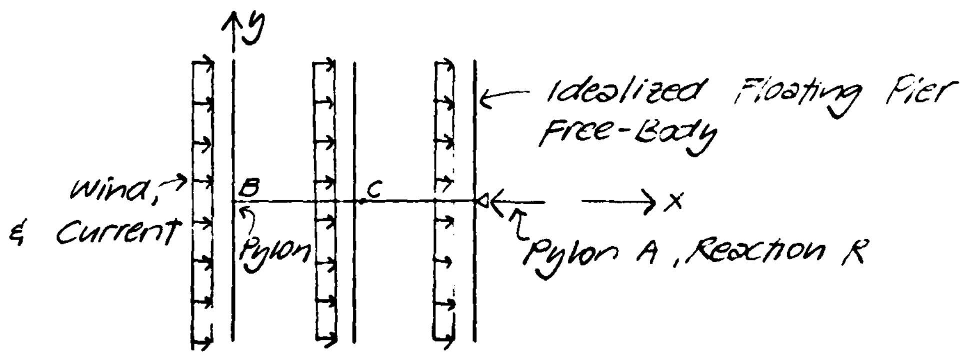
Cont' Wind Loading

At point C

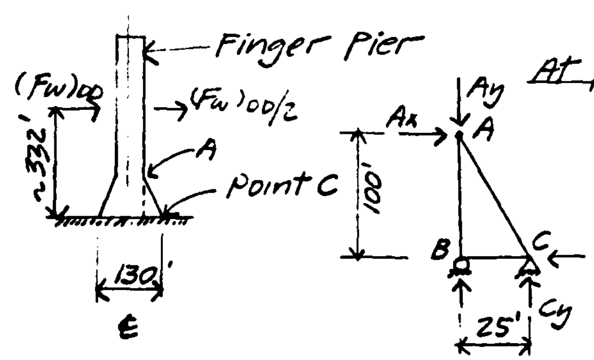
Total wind $Load_c$ = Σ Tributary $(Fw)_{OO}$ + $(Fw)_{F.P.}$

a) Global Axial x @ Point C :



Total Axial $P_{wx,C} \simeq 2(Fw)_{OO} + 2(Fw)_{OO}/2$
$= 2 \times 144 + 2 \times 72$
$= 432^K$ ←

b) Global Axial y @ Point C : Due to Finger Bending



At point A

$Ay = \frac{(144+72) \times (332-100)}{80}$
$= 626^K$
$Ax = (144+72)/2$
$= 108^K$

$\Sigma M_B = 0$ ∴
$Cy = 108 \times 100/25 = 432^K$

Total Axial $P_{wy,C} = 2 \times 432^K$
$= 864^K$ ←

At Pylon A & B

Axial @ Pylons $= [6(144+72)+20]/2 = \underline{658^K}$ ←

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PROJECT: Navy Pier
ITEM: Floating Marina Pier, Sch. 2
DESIGN: Environmental Loads: Current
DATE: 11/83 AM

SHEET: A-3.12
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Current loading in x-x direction

Assume : a) Use same assumptions for wind loading
b) 3 knots current
c) Ships at leeward side will pick 33% of current force on those in windward side.
d) All vessels are secured to mooring

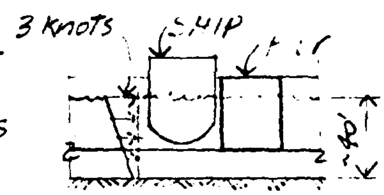
From new DM 26, $F_c = C_{yc}(1/2)\rho_c V_c^2 (LWL) T$

C_{yc} from Fig. 4 : when $\frac{W.D.}{T} = \frac{40}{29}$ $\frac{\text{Water depth}}{\text{Draft}}$
$= 1.4$

Then $C_{yc} \cong 1.9$

$\rho_c = 1.9876$ #-sec²/F⁴ @ 68°F

$\overline{V_c}$ = avg. current on hull, F/s
$= (3 + 0.83)/2$ $\quad 3^k - 40'$
$= 1.91$ knots $\quad x - 11'$
$= 3.22$ F/s



LWL = Length @ Water line, Ft.
T = Draft, Ft.

For DD-963

$(F_c)_{DD} = 1.9 \times \frac{1}{2}\, 1.9876 \times 3.22^2 \times 530' \times 29'$
$= \underline{301}$ ←

$0.33\,(F_c)_{DD} = \underline{\underline{99^k}}$ ←

For Floating Pier : $T = 25'$ $\quad W.D/T = 40/25$
$= 1.6$

Then $C_{yc} = 1.8$
$\overline{V_c} = 2.1$ knots
$= 3.55$ F/s

LWL = length @ Water Line, Ft.
T = Draft, Ft.

$(F_c)_{F.P.} = 1.8\,(1/2)(1.9876)(3.55)^2(200')\times \overline{25'} = \underline{\underline{113^k}}$ ←

Cont' Current loading in x-x direction :

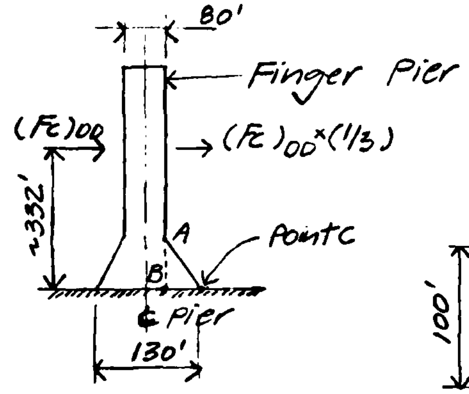
At point C

Total Current Load$_C$ = Σ Tributary $(Fc)_{OO}$ + $(Fc)_{F.P.}$

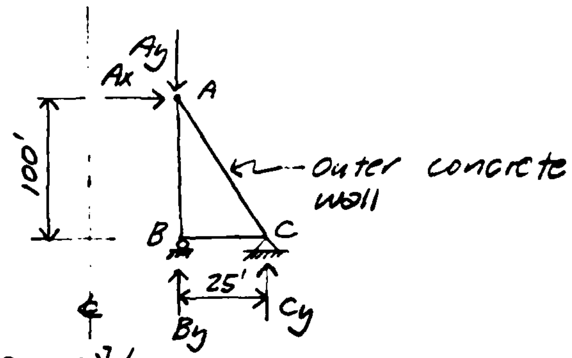
a) Global Axial x @ Point C :

$$\text{Total Axial } P_{Cx,C} = 2(Fc)_{OO} + 2(Fc)_{OO}$$
$$= 2(301) + 2(99)$$
$$= \underline{\underline{800^k}} \leftarrow$$

b) Global Axial y @ Point C : Due to bending in Finger



$(Fc)_{OO} = 301^k$



At Point A :

$$A_y = [(301 + 99) \times (332 - 100)]/30$$
$$= 1110^k$$

$$A_x = (301 + 99)/2$$
$$= 200^k$$

$$\Sigma M_B = 0 \quad \therefore \quad C_y = 200^k \times 100'/25'$$
$$= 800^k$$

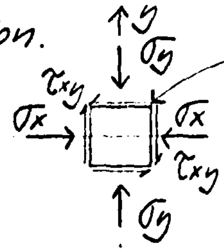
Total Axial P_y @ Point C : $2 \times 800 = \underline{1600}^k \leftarrow$

At Pylon A and B :

$$\text{Total Axial } P_{Cx,B} = [6(301 + 99)/2 + 113^k$$
$$= \underline{\underline{1313^k}} \leftarrow$$

Primary Stresses at Point C: due to Environmental loads in x-x direction.

1) wave
2) wind and
3) current



Point C: Bottom slab (x-y plane) near bulkhead

State of Principal Stresses @ C

1) Wave induced stresses (at Section A, through Pt. C)

$M_{max} = 2136000^{K\text{-}F}$ (due to Hogging)

$\sigma_{TOP,x} = 2136000^{K\text{-}F} / 11173^{F^3} = 191^{KSF} = 1328^{psi}$ (T)

$\sigma_{BOT,x} = 2136000/10971 = 195^{KSF} = 1352^{psi}$ (C) ⟵

2) Wind x-x Induced Stresses : @ Point C

Axial x @ C : $\sigma_x = P_{wx}/A_y$ A_y = Transverse Cross Section
$= 482 \times 2$
$= 964^{F^2}$

$\sigma_x = 432/964$
$= 0.45^{KSF} = 3^{psi}$ (C) ⟵

Axial y @ C : $\sigma_y = P_{wy}/A_x$ A_x = Longitudinal Cross-Sect.
$= 864^K / 187^{SF}$* Assume axial Load is distributed in a width of 40' only.
$= 4.6^{KSF}$
$= 32^{psi}$ ⟵ (C)
$A_x = (18+10+15)''/12 \times 40' + 1' \times 44'$
$= 187\ \phi$

3) Current x-x Induced Stresses :

Axial x : $\sigma_x = P_{cx}/A_y$ $P_{cx} = 800^K$
$= 800/964$
$= 0.83 = 6^{psi}$ ⟵

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DESIGN:	State of Stresses @ Point C
DATE:	1.1.85 KM
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Cont' Current Induced Stresses :

Axial y : $\sigma_y = P_{cy}/A_x$ $A_x = \phi$, $P_{cy} = 1600^K$

$\sigma_y = 1600/187$

$= 8.6 \times 1000/144$

$= 59$ psi ⟵

Total Primary Stresses @ Point C due to Environmental Loads :

$\Sigma \sigma_x = 1352 + 3 + 6$

$= 1361$ psi ⟵

$\Sigma \sigma_y = 32 + 59$

$= 91$ psi ⟵

$\Sigma \tau_{xy} \simeq 0$ due to Global Loading

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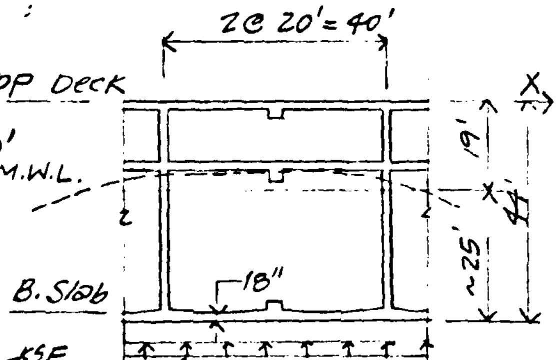
Secondary Stresses @ Pt.C :

1) Buoyancy Bending$_{yy}$:



Longitudinal Slab Span: 20'

t_{slab} = 20" @ support

a) Static Pressure :

D.L:

Concrete: $\overline{18}''/12 \times .150 = 0.225^{KSF}$

Mech. + Misc. $= 0.05$

Static Buoy: $\overline{27} \times .062 = \underline{-1.67}$

Resultant : -1.4^{KSF}

Buoyancy @ 25'+1.5' = 26.5'

wave ht.

Dynamic Pressure : Does not control in this case because Sea State 2 is mild.

Slab Section Properties at Pt. C

Haunched Slab, $t = 20''$

$t_{midspan} = 16''$ $\quad I = 12 \times 20^3/12 = 8000''^4$

$S = 800''^3$

b) Moment : $L_n \cong 19'$, $L_1 = 40'$

$L_1/L_n > 2 \rightarrow$ One-way Slab

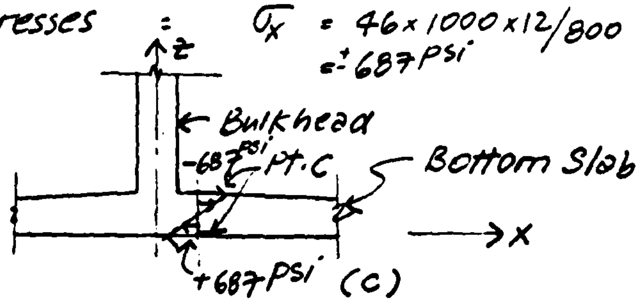
$M^-_{max} = wL_n^2/11$

$= 1.4 \times 19^2/11$

$= 46^{k\text{-}F}$

c) Stresses : $\sigma_x = 46 \times 1000 \times 12/800$

$= \pm 687^{psi}$



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315 Bay St., San Francisco, Ca. 94133

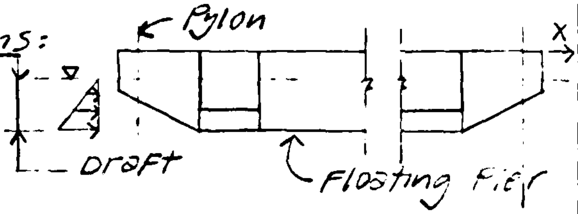
PROJECT:	Navy Piers
ITEM:	Floating Marina Pier Sch. 2
DESIGN:	Stresses @ Pt. C: Buoyancy
DATE:	11/83 KM
SHEET:	A-27
OF	
REVISION:	

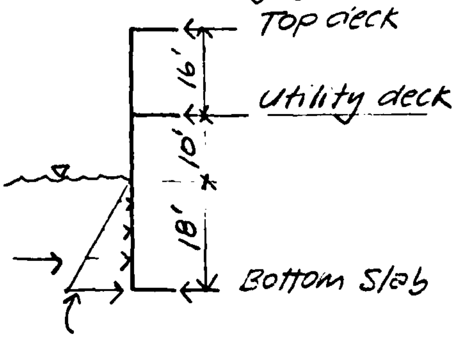
Cont' Buoyancy Secondary Stresses @ Point C

2) Buoyancy Axial Global Stresses:

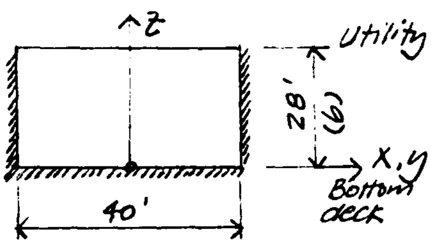
a) Dead Load only: Draft ≃ 18'

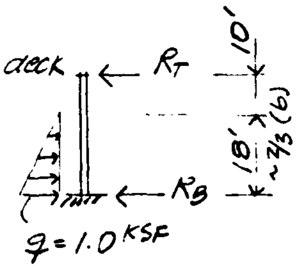
In x-x and y-y directions:





$P_B = 0.062 \times 16'$
$= 1.0$ KSF





From Formulas for Stress and Strain by Roark & Young,

Case no. 10 dd : $a/b = 40'/28' = 1.43$ ∴
$\gamma_1 \cong 0.29$

At $x=0$, $z=0$ → $R_B = \gamma_1 q b$
or $y=0$ $= 0.29 \times 1.0 \times 28'$
$= 8.1$ k/' or 8 k ←

Stresses @ Pt. C :

$\sigma_{x,y} = R_B/A$ $A =$ Unit Area
$= 8 \times 1000/240$ $= 12'' \times 20'' = 240''^2$
$= 33$ psi ←

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PROJECT: Navy Piers
ITEM: Floating Marina Pier, Sch. 2
DESIGN: Stresses @ P.t. C: Buoyancy
DATE: 11/33 KM

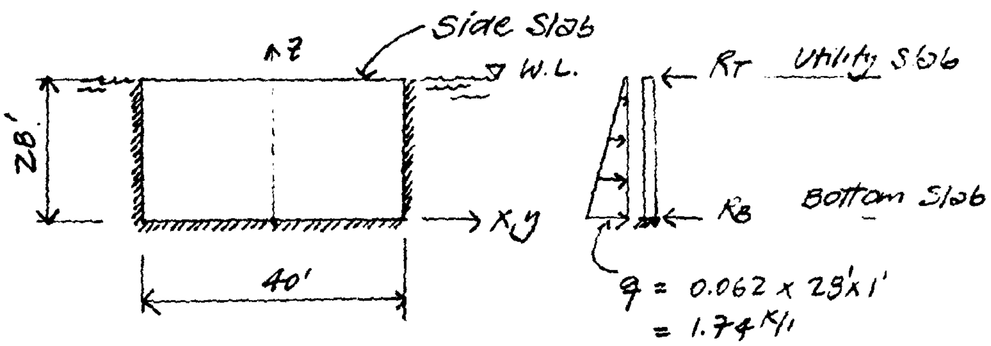
SHEET: A-2.18
OF
REVISION:

Cont' Axial Global Stresses due to Buoyancy:

b) Dead + Live + Tide:

Total Water Head = 25' + 2' = 27' Say 28'

D+L — 25', Tide, wave — 2'



$q = 0.062 \times 28' \times 1'$
$= 1.74^{K/'}$

Case no. 9d, At $x=0$ and $z=0$
or $y=0$

$a/b = 1.43 \rightarrow \gamma_1 = 0.376$

$\therefore \quad R_B = \gamma_1 q b$
$= 0.376 \times 1.74 \times 28'$
$= 18.3^{K/'}$

Stresses @ Point C:

$\sigma_{x,y} = R_B/A$
$= 18300^{\#}/240$
$= 76\ psi$

A = Unit Area
$= 12'' \times 20'' = 240''^2$

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PROJECT: Navy Piers
ITEM: Floating Marina Pier, Sch. 2
DESIGN: Summary of Stresses @ C
DATE: 11 83 RM

SHEET: A-2.19
OF
REVISION:

Summary of Stresses @ Point C : Compression (-)
Tension (+)

Loading	Stresses @ Point C (psi)					
	Local		Global		Σ	
	σ_x	σ_y	σ_x	σ_y	σ_x	σ_y
Dead + Live + Buoyancy	± 687	—	-76	-76	-687	-76
$Wave_{x-x}$ (Hogg.)	—	—	-1352	—	-1352	—
$Wind_{x-x}$	—	—	-3	-32	-3	-32
$Current_{x-x}$	—	—	-6	-59	-6	-59
					-2048 psi	-167 psi

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PROJECT: Navy FOIS
ITEM: Floating Marina Pier Sch. 2
DESIGN: X-X Prestressing
DATE: 11 85 KM

SHEET: A-3.20
OF
REVISION:

Prestressing in X direction :

Partial Prestressing : Assume 80% of Prestress to offset tensile stresses at Bottom due caused by wave Action (Sagging)

$\sigma_{bottom} = +1352^{psi}$

$Prestress = -0.80 \times 1352^{psi}$
$= -1082^{psi}$

Total Prestressing Force :
$= 1082 \times 418 \times 2 \times 144/1000$
$= 130207^{K}$

Assume 270 Grade Prestressing Strands :

$A_{required} = 130207^{K}/270 \times 0.70^{ksi}$
$= 689^{in^2}$
$= 4.8^{ft^2}$

Total % Steel in Gross-Area @ Point C :
Mild = 1% (assume)
presst. = $4.8/2 \times 418 = 0.006$
$= 0.6\%$
$\Sigma = 1.6\% \longrightarrow 13.4^{\phi} \longleftarrow$

Transformed Area of concrete @ Point C :

$E_c \cong 120^{1.5} \times 33\sqrt{7000} \qquad E_s = 29 \times 10^6$ psi
$= 3.7 \times 10^6$ psi

$A_T = 936^{\phi} + 13.4(29/3.7)$
$= 1038^{\phi}$ or 11% increase in Area

$Prestress = -0.80 \times 1352/1.11$
$= -974$ psi $\longleftarrow$

Wave load $= 1352/1.11$
$= \pm 1218$ psi $\longleftarrow$

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PROJECT: Navy Piers
ITEM: Floating Marine Pier, Sch. 2
DESIGN: Final Stresses @ Pt. C
DATE: due to x-x loading

SHEET: A-2.21
OF
REVISION:

Cont' prestressing in x-direction @ Pt. C :

Residual Tension = above (Sag.) - prestress
= 1218 psi − 974 psi
= 244 psi ⟶ Use mild steel

Final compressive stresses at point C : x-x loading

$\sigma_{x,max} = \Sigma\sigma_x + \sigma_x$ prestress , includes transformed area
$= \pm 687 \pm 1218 - 3 - 6 - 974$
$= -2888$ psi ⟵

$\sigma_{y,max} = \Sigma\sigma_y + \sigma_y$ prestress , assume 400 psi
$= -76 - 32 - 59 - 400$
$= -567$ psi ⟵

$\tau_{xy} \simeq 0$ at Bott. slab extreme fibers

Principal Stress at C :

$\tau_{max,min} = \pm \frac{1}{2}\sqrt{(\sigma_x - \sigma_y)^2 + (2\tau)^2}$

$= \pm 1/2 \sqrt{(-2888+567)^2 + 0}$

$= \pm 1161$ psi

$\sigma_{max,min} = 1/2(\sigma_x + \sigma_y) \pm \tau(max)$
$= 1/2(-2888 - 567) \pm 1161$
$= -2889$ psi, -567 psi ⟵

f'_c required $= (\sigma max/0.45)$
$= [2889/0.45]$
$= 6420$ psi

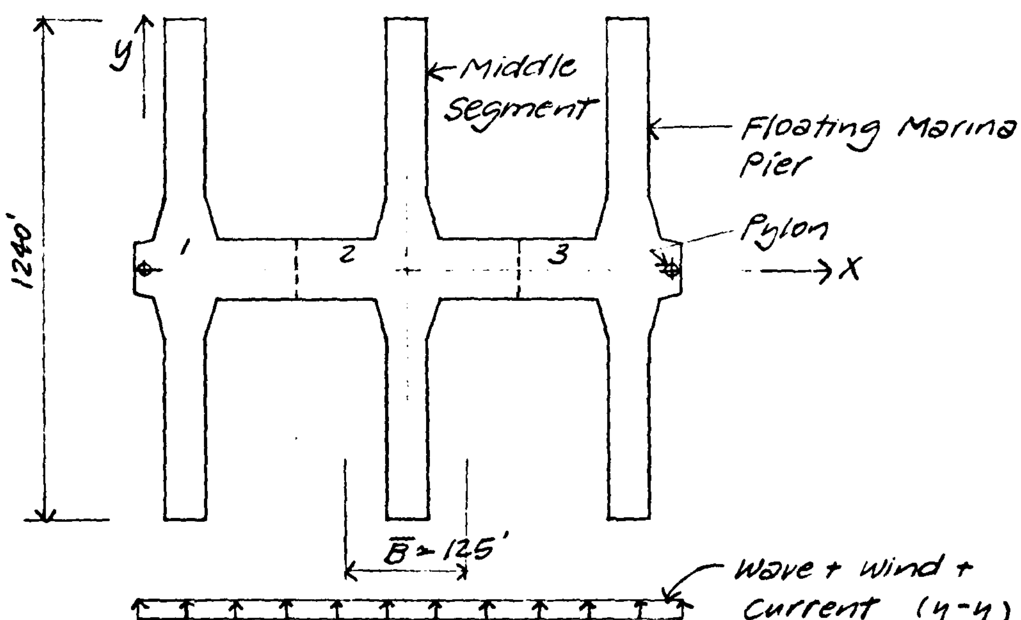
Use Lightweight concrete $f'_c = 7000$ psi ⟵

Environmental Loading in y-y direction :

Wave Loading :

ABS Wave Moments and Shears @ Sea State 2 :

For wave loading in y-y direction one can subdivide the Floating pier in 3 segments. Each segment contains the two opposite Fingers plus the portion of the spine that connects them. Using this assumption then the critical segment would be the widest one which is #2 below.



Wave Moment at Middle Segment (2) :

$Mt_{yy} = Ms_{yy} + Mw_{yy}$

Ms_{yy} = Still water Bending Moment
= 0 ∴ assume relatively uniform D.L & L.L distribution, tendon eccentricity will balance any possible still water Moments.

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PROJECT: Navy Piers
ITEM: Floating Marine Pier, Sch. 3
DESIGN: Env. Loading in y-y dir.
DATE: III 83 KM

SHEET: A-2.33
OF
REVISION:

Cont' wave moments$_{yy}$ at Segment 2

Mw$_{yy}$ at Middle Segment (2)

$M_{w_{yy}} = C_2 L^2 \bar{B}\, He\, K_6$, Max. wave Moment @ $y=0$

$K_6 = 1.0$ For $C_6 > 0.80$

$C_6 = 1.0$ assumed

$C_2 = 7.1 \times 10^{-4}$

$L = 1240'$

$\bar{B}_{y=80'} = [440' \times 80' + 100' \times 100' + 80' \times 410'] / 620'$

$= 125'$

$He = 26.75'$ $1000' < L \leq 1400'$

$M_{w_{yy}}$ at Spine $= 7.1 \times 10^{-4} \times 1240'^2 \times 125' \times 26.75' \times 1.0 \times 2.24$

$= 8.18 \times 10$ k-F

Maximum moment that can occur at Segment 2 (Hogg.) Force due to 3' high waves :

$M_{max,w_{yy}}$ at spine centerline $= \frac{H}{He} \times M_{w_{yy}}$

$\frac{3'}{26.75'} \times 8.18 \times 10^6$ k-F

$= \underline{\underline{9.2 \times 10^5 \text{ k-F}}}$ ←

$\sigma_{w\,max}$ = Maximum allowable wave bending stress :

$= 0.45\, f'c/2$ $f'c = 7000$ psi (assumed througnT)

$= 1575$ psi

ABS Wave Bending Moment and Shear Distribution :

Wave in y-y direction

$M_{max_{yy}} = 9.2 \times 10^5$ K-F @ centerline $L = 1240'$

Stations at 124'-0" along y axis

Station	Bending Moment (Wave Height = 3')			
0	0	=	0	K-F
2	$0.10 \times 9.2 \times 10^5$	=	92000	
4	$0.35 \times 9.2 \times 10^5$	=	322000	
6	$0.68 \times 9.2 \times 10^5$	=	625600	
8	$0.95 \times 9.2 \times 10^5$	=	874000	
centerline Moment → 10	$1.00 \times 9.2 \times 10^5$	=	920000	
12	$0.94 \times 9.2 \times 10^5$	=	864800	
14	$0.74 \times 9.2 \times 10^5$	=	680800	
16	$0.43 \times 9.2 \times 10^5$	=	395600	
18	$0.13 \times 9.2 \times 10^5$	=	119600	
20	0	=	0	

Shear Distribution

$Fmax_{yy} = Fs_{yy} + Fw_{yy}$

Fs_{yy} = still water shear
= 0

$Fw_{yy} = K\,Mw/L$

Mw = Max. wave moment
= 920000 K-F

$L = 1240'$

$K = 0$, ends
$= 2.6$, $186' < y < 372'$
$= 1.6$, $496' < y < 682'$
$= 2.5$, $806' < y < 992'$

$Fw_{yy}(K=0) = 0$
$(K=2.6) = 1929^k$
$(K=1.6) = 1187^k$
$(K=2.5) = 1855^k$

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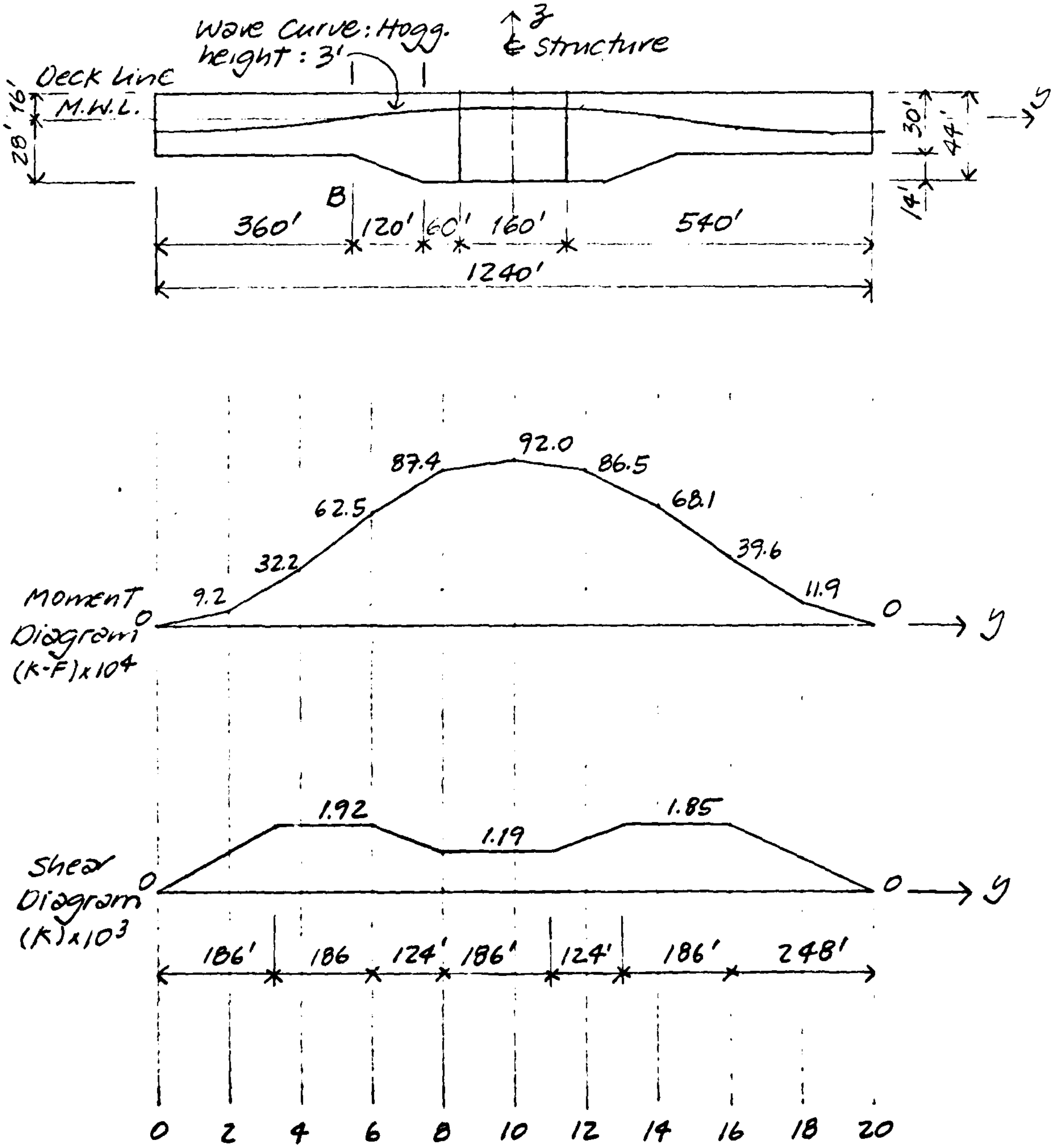
PROJECT:	Navy Piers
ITEM:	Floating Marina Pier Sch 2
DESIGN:	Envir. Loads in y-y direct
DATE:	11/83 KM

SHEET: A-2.55
OF
REVISION:

Wave Moments and Shear Envelopes (y-y direction)



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PROJECT: Navy Piers
ITEM: Floating Marina Pier Sch. 2
DESIGN: Envir. Loads in y-y dir.
DATE: 11/83 XM

SHEET: A-2.26
OF
REVISION:

Check Finger Pier Section For Wave Bending

Section at B : $F'_c = 7000$ psi

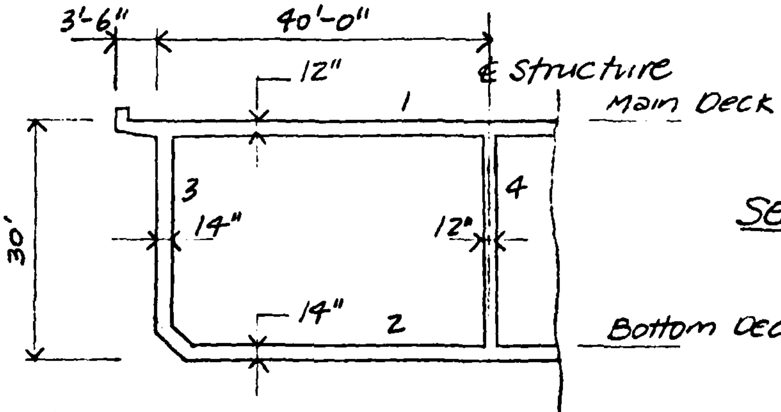
M_{wyy} at B = 322000 + (625000 − 322000) × 112'/124'
= 595677 K-F

Shear Force$_B$ = 1920 K

σ_{max} @ B = Maximum allowable bending stresses
= 0.45 × 7000/2
= 1575 psi

Minimum St,b required at (B)

St,b = 595677/(1575 × 144)/1000
= 2626 F³ ←



SECTION (B)

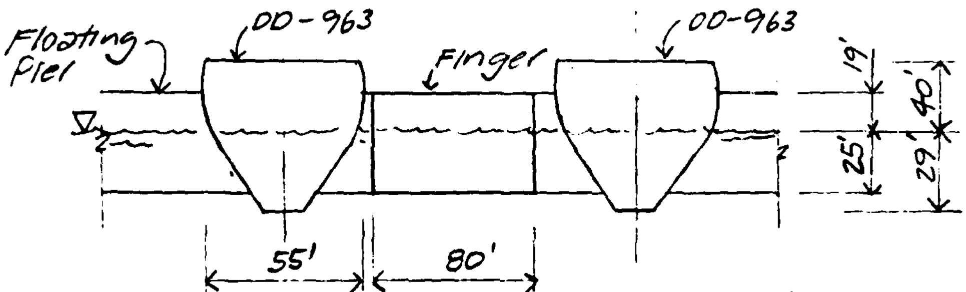
	A (ϕ)	y'	Ay (F³)	Ay² (F⁴)	Iox (F⁴)	yt / yb	St / Sb
1	43.5	29.5	1283	37856	4		
2	46.7	0.58	27	16	5		
3	32.5	15.1	490	7402	2096		
4	13.9	15.1	210	3173	898	15.3	2867
	136.6	14.7	2010	48447	3003	14.7	2984

$I_{ox} = [48447 + 3003 - 136.6 \times 14.7^2] \times 2$
$= 43864$ F⁴

$St = 43864/15.3 = 2867$ F³ > 2626 F³ ✓ ←

Wind Loading to Pylons in the y-y direction

Assume :
a) Sea State 2 : wind velocity of 50 MPH in y-y direction @ 0° to longitudinal axis of ships.
b) All (12) ships are nested in the Pier
c) Ships nested in Leeward side of every Finger takes 50% of wind loads on ships at the windward side.
d) Only DD-963 nested at the pier
e) water does not absorbed energy



$A_{DD} = 55' \times 40' = 2200^{\square}$ $A_F = 80' \times 19' = 1520^{\square}$ $A_{DD} = 2200^{\square}$ $A_P = 19'(1000 - 6 \times 55' - 3 \times 80') = 8170^{\square}$

Total area exposed to wind:

$A_T = 6(A_{DD}) + 3(A_F) + 6/2(A_{DD}) + A_P$
$= 6 \times 2200 + 3 \times 1520 + 3 \times 2200 + 8170$
$= 32530^{\square}$ say $34000^{\square}$, to include higher portions of vessels

From DM 26 : $F_W = C_{yw} \times 1/2 \rho_w V_w^2 A_s$

$C_{yw} = 1.0$, wind at 0° to ships
$\rho_w = 0.00237^{\#}\text{-sec}^2/F^4$ @ 68°F
$V_w = 50$ MPH $= 73$ F/sec @ 40' above waterline

$F_W = 1.0 \times 1/2 \times 0.00237 \times 73^2 \times 34000^{\square}$
$= 214^k$ total

Wind Load/Pylon $= 214^k/2 = \underline{107^k}$ ←

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Current Loads in y direction :

Use same assumption as current Forces in x-direction.
Current velocity : 3 knots at surface.

$Area_c$ DD-963 = 29'x 50'
= 1450 ☐'

$Area_c$ Finger = 25'x 80'
= 2000 ☐'

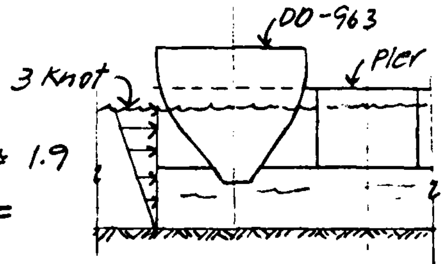
$Area_c$ Spine = 25'(1000' - 6x 50' - 3x 80')
= 11500 ☐'

Total Area exposed to current :

$A_{cT} = 6(A_{DD}) + 6(0.5 A_{DD}) + 3(A_F) + A_{spine}$
$= 6 \times 1450 + 6 \times 0.5 \times 1450 + 3 \times 2000 + 11500$
$= 30550$ ☐' say 31000 ☐'

New DM 26.6 (Pg. 26.6-3)

$F_C = \Sigma C_{yc} (1/2) \rho_c V_c^2 A_c$



When $\frac{W.D.}{T} = \frac{40}{29}$ ($\frac{\text{water depth}}{\text{(Draft)}}$)
= 1.4 Then $C_{yc} \doteq 1.9$

ρ_c = 1.9876 #-sec²/F⁴ @ 68°F
$\bar{V}_{c_s}$ = avg. current, F/s
= (3 + 0.83)/2
= 1.9 knots = 3.22 F/s @ ships

$\bar{V}_{c_p}$ = (3 + 1.43)/2   3 knots → 40'
= 2.21 knots   x → 19'
= 3.74 F/s @ Pier

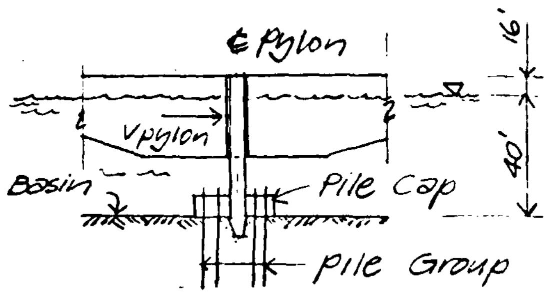
W.D./T = 40/25 = 1.6 Then $C_{yc\,pier}$ = 1.7

Total $Force_c$ = (1/2)(1.987)[1.9 x 3.22² x 13050 ☐' + 1.6 x 3.74² x 17500 ☐']
= 645^k

Total Current Load to each pylon : ~ 645/2 = 323^k ←

Total Environmental Load to Pylon in y-direction:

$$V_{pylon_y} = V_{wind_y} + V_{current_y}$$
$$= 107 + 323$$
$$= \underline{\underline{430^k}}$$



Total Environmental Load to Pylon in x-x direction

$$V_{pylon_x} = V_{wind_x} + V_{current_x}$$
$$= 658^k + 1313^k$$
$$= \underline{\underline{1971^k}} \longleftarrow \text{Governs}$$

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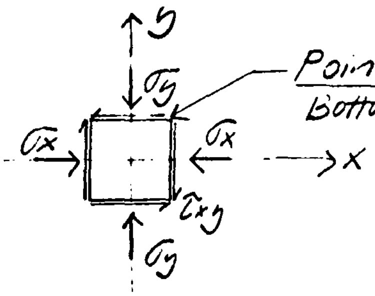
PROJECT: Navy Piers	SHEET: A-2.20
ITEM: Floating Marine Pier, Sch. 2	OF
DESIGN: State of Stresses @ PT. B	REVISION:
DATE: 11/85 KM	

Primary Stresses at Point B at Finger of Segment 2:



Point B at Section B:
Bottom slab of Finger, Segment 2

*Due to Environmental Loads in y-y direction:

1) Wave Induced Stresses:

$M_{max} = 595677^{K\text{-}F}$ (Wave Hogging)

$\sigma_{top,y} = 595677 \times 1000 / 2867 \times 144$
$= 1443^{psi}$ (T) ←

$\sigma_{bott,y} = 595677 \times 1000 / 2984 \times 144$
$= 1386$ psi (C) ←

2) Wind Induced Stresses:

Assume that wind and current loads acting on windward ships and pier are transfered through moorings located at the c.g.c. of Finger Pier. That is no torsion at the fingers due to these loads.

$2(F_{W\,DD}) = 2(1.0 \times 1/2 \times 0.00237 \times 73^2 \times 2200^{\#})$
$= 27.8^k$

$(F_W)_{Fing} = 1.0 \times 1/2 \times 0.00237 \times 73^2 \times 1520$
$= 9.6^k$

$\Sigma F_W = 27.8^k + 9.6^k$
$= 37.4^k$

$\sigma_{wy} = F_W / A_{F@B}$
$= 37.4 \times 1000 / 273^{\#} \times 144 = 1^{psi}$ (small)

*Must check Section B for loads in x direction.

Cont' Primary stresses at Point B

3) Current y-y induced stresses:

$$2(Fc_{yy})_{DD} = 2[1.9 \times (1/2) \times 1.987 \times 3.22^2 \times 1450]$$
$$= 57^K$$

$$(Fc_{yy})_{Fing.} = 1.6 \times 1/2 \times 1.987 \times 3.74^2 \times 2000^{\phi}$$
$$= 44.5^K$$

$$\Sigma Fc_{yy} @ B = 57 + 44.5$$
$$= 101.5^K$$

$$\text{Stress } \sigma_{cy} = 101.5 \times 1000/273 \times 144$$
$$= 3 \text{ psi}$$

Total Primary stresses at Point B at Fingers due to environmental loads:

$$\Sigma \sigma_y = 1386 + 1 + 3$$
$$= 1390 \text{ psi}$$

$$\Sigma \tau_{xy} \equiv 0^{psi} \text{ at slab edge}$$

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PROJECT:	Navy Piers
ITEM:	Floating Marina Pier, Sch. 2
DESIGN:	Stresses at PT. B (Finger)
DATE:	11/ 82 AM

SHEET: A-...
OF
REVISION:

Secondary stresses at Pt. B, Finger at Segment 2:

1) Buoyancy Bending σ_y

At Bottom Slab: Span = 20'-0"
$t_{support}$ = 14"

a) Static Pressure:

D.L:

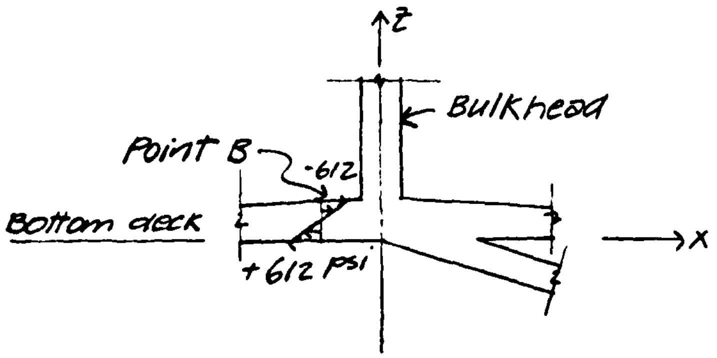
Concrete: $14/12 \times .15 = 0.175^{KSF}$
Mech. + Misc. $= 0.05$
Buoyancy: $\overline{13} \times .062 = -0.81$
Resultant: $= -0.59^{KSF}$

Slab Section Properties at Point B

$I = 12 \times 14^3/12 = 2744''^4$
$S = 2744 \times 2/14 = 392''^3$

b) Moment: $L_n \simeq 19'$, $L_1 = 40'$
$L_1/L_n > 2 \rightarrow$ One-way slab
$\bar{M}_{max} = wL_n^2/11$
$= 0.59 \times 19^2/11$
$= 20^{K-F}$

c) Stresses: $\sigma_{B,y} = 20 \times 1000 \times 12/392$
$= \pm 612$ psi $\leftarrow$

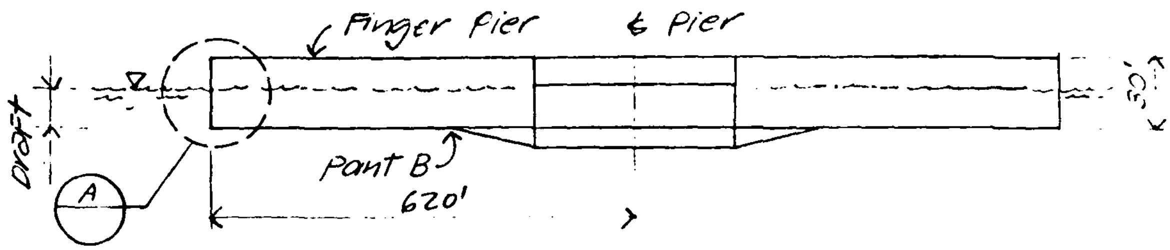


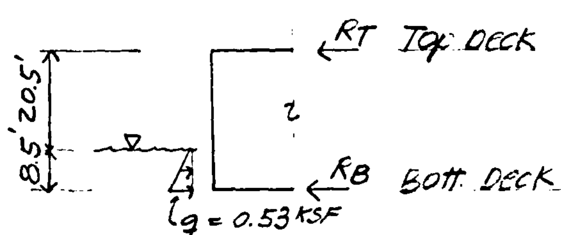
T.Y.LIN INTERNATIONAL STRUCTURAL ENGINEERING 315 Bay St., San Francisco, Ca 94133	PROJECT: Navy Piers	SHEET: A-2.35
	ITEM: Floating Marine Pier, Sch. 2	OF
	DESIGN: Stresses @ Point B: Finger	REVISION:
	DATE: 83 KM	

Cont' Buoyancy Secondary Stresses @ Pt. B

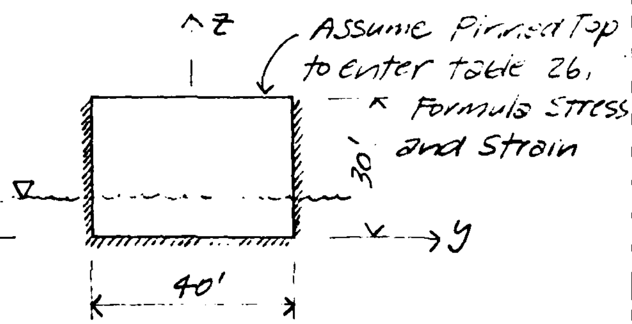
2) Buoyancy Axial Global Stresses:

a) Dead Load only : Draft ≅ 5'





DETAIL A



Case 9 add at x = 0, z = 0

$R_B = \delta_1 q b$ $\quad a/b = 1.33 \quad \delta_1 \cong 0.11$ (extrapolating)

$R_B = 0.11\ (0.53)(30)$

$= 1.83^K/F$ or 1.8^K

Stresses at Point B :

$\sigma_{x,y} = 1800^{\#} / 12'' \times 14''$

$= 11$ psi ⟵ Dead Load draft

b) Dead + Live Load (300 PSF) + Tide : $12.5' + 2' = 14.5$ Say $15'$

$\delta_1 = 0.20$

$q = 0.062 \times 15.0' = 0.93$ KSF

$R_B = 0.20 \times 0.93 \times 30'$

$= 5.6\ ^K/F$ or 5.6^K ⟵ Full draft + Tide

Stresses at Point B : $\sigma_{x,y} = 5600/12 \times 14 = 33$ psi ⟵

Stress Summary at Point C due to Environmental Loads in the y direction.

Loading	Stresses at Pt. B (psi)					
	Local		Global		Σ	
	σx	σy	σx	σy	σx	σy
Dead + Live + Buoyancy	—	± 612	-33	-33	-33	+ 579 - 645
Wave$_{yy}$ (Hog.)	—	—	—	-1386	—	-1386
Wind$_{yy}$	—	—	—	—	—	—
Current$_{yy}$	—	—	—	- 3	—	- 3
				Σ	- 33	-2304 psi

Prestressing in y direction

Partial Prestressing : Assume 80% of prestress to offset tensile stresses at bottom slab due to wave sagging.

σ_{bottom} @ B = + 1386 psi

Prestress = −0.80 × 1386 psi
= −1109 psi

Total Prestressing Force F,
F = 1109 × 137 × 2 × 144 / 1000
43749 K

Assume 270 Grade Prestressing Strands:

Areq'd = 43749 K / 270 × 0.70 ksi
= 231 in²
= 1.6 ft²

Total % Steel in Cross-Area @ Point C :

Assume mild steel = 1%
Prestressing = 1.6 / 2 × 137 = 0.006
= 0.6%
Σ = 1.6% ⟶ 4.4 ft²

Transformed Area of Concrete @ Pt. B of Finger :

f'c = 7000 psi , Ec = 3.8 × 10⁶ psi
A_T = 2 × 137 − 4.4 + 4.4 × 29 / 3.8
= 303 ft² or 11% increased area

$\sigma_{prestress_y}$ = −0.80 × 1386 / 1.11
= −999 psi ⟵

Wave Load$_y$ @ Bottom = 1386 / 1.11
= ±1249 psi ⟵

Residual Tension: Wave Sag$_y$ − Prestress$_y$
= 1249 − 999 = 250 psi **Use mild steel**

Final Compressive Stresses at Pt. B due to y-y loads:

$\sigma_x max_B = \Sigma\sigma_x + Prestress_x$
$= -33 + 0$ at Pt. B
$= -33$ psi

$\sigma_y max_B = \Sigma\sigma_y + Prestress_y$ at B
$= -645 - 1249 - 3 - 999$
$= -2896$ psi

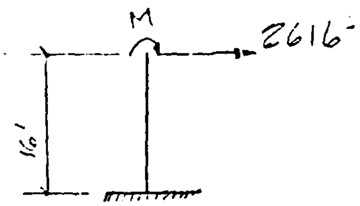
F'c required = $\sigma max./0.45$
= 2896/0.45
= 6435 psi

Use Normalweight Concrete, F'c = 7000 psi at Fingers and Spine.

TYLIN INTERNATIONAL STRUCTURAL ENGINEERING 315 Bay St., San Francisco Ca 94133	PROJECT: Navy Piers	SHEET: A-2.57
	ITEM: Floating Marina P..r, Sec 2	OF
	DESIGN: [illegible]	REVISION:
	DATE: [illegible] R.G.	

Pylon Design : Assume Pylon pinned @ the top.

Load transverse ... = 1971 × 1.05
= 2093^k



Ultimate Strength Design :

$P_u = 1.25(2093) = 2616^k$

$M_u = (2616)(16) = 41856^{k\text{-}ft}$

preliminary investigation ⇒ try 13'0" O.D./11'-0" ϕ I.D.
Pylon w/ 2.5% steel $f'_c = 6000$ psi.

$A_s = 1$ in^2 or ~140 # 9 bars

distance to reinforcing steel
assume bars spaced equally

the output from ROUNDC gives the distance from the compression face to each pair of reinforcing bars
The steel stress is also given. (see next page)

For the output shown, the depth of compression block = 15.5 in

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PROJECT:	Navy [illegible]
ITEM:	Floating Marinas [illegible] Scheme 2
DESIGN:	Pylons
DATE:	11/3? KM

SHEET: A-2.35
OF
REVISION:

'Cont' Pylon Design :

DIST. TO COMP. FACE	STRESS
4.09	+60.000
4.22	+60.000
4.52	+60.000
4.97	+60.000
5.56	+60.000
6.30	+60.000
7.13	+56.747
8.21	+52.431
9.37	+47.523
10.68	+42.034
12.12	+35.973
13.69	+29.354
15.39	+22.190
17.22	+14.495
19.17	+ 6.285
21.24	- 2.423
23.42	-11.613
25.71	-21.265
28.11	-31.361
30.61	-41.880
33.20	-52.800
35.89	-60.000
38.66	-60.000
41.51	-60.000
44.43	-60.000
47.42	-60.000
50.47	-60.000
53.57	-60.000
56.73	-60.000
59.93	-60.000
63.16	-60.000
66.43	-60.000
69.72	-60.000
73.02	-60.000
76.34	-60.000
79.65	-60.000
[illegible]	[illegible]

DIST. TO COMP. FACE	STRESS
82.97	-60.000
86.27	-60.000
89.56	-60.000
92.83	-60.000
96.06	-60.000
99.26	-60.000
102.42	-60.000
105.52	-60.000
108.57	-60.000
111.56	-60.000
114.48	-60.000
117.33	-60.000
120.10	-60.000
122.79	-60.000
125.38	-60.000
127.88	-60.000
130.28	-60.000
132.57	-60.000
134.75	-60.000
136.82	-60.000
138.77	-60.000
140.60	-60.000
142.30	-60.000
143.87	-60.000
145.31	-60.000
146.62	-60.000
147.78	-60.000
148.81	-60.000
149.69	-60.000
150.43	-60.000
151.02	-60.000
151.47	-60.000
151.77	-60.000
151.91	-60.000

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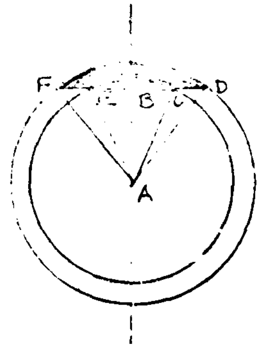
PROJECT: Nova [illegible]		SHEET: A-239
ITEM: [illegible]		OF
DESIGN: Pylons		REVISION:
DATE: 11/3? RM		

Pylon Design



AE = 62.5"
AC = 66"
AD = 78"

$\angle EAD = \arccos \frac{62.5}{78} = 36.75° \approx .641$ rad

$\angle EAC = \arccos \frac{62.5}{66} = 18.74° \approx .327$ rad

$\angle CAD = 36.75 - 18.74 = 18.01° = .314$ rad

Area sector ADF = $(.641)(78)^2 = 3900$ in²

Area sector ACE = $(.327)(66)^2 = 1424.4$ in²

height of $\Delta ACD = (66)\sin 18.01 = 20.4$ in

Area $\Delta ACD = (½)(20.4)(78) = 795.6$

Area comp. block = $3900 - 1424.4 - (795.6)(2) = 884.4$

c.g. comp. block

$$= \frac{\left[\frac{(2)(78)\sin 36.75}{(3)(.641)}(3900) - \frac{(2)(66)\sin 18.74}{(3)(.327)}(1424.4) - (795.1)(2)(2/3)(62.5)\right]}{884.4}$$

$= 73.45"$

$C = (884.4)(.85)(6) = 4510$

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PROJECT: NSVM Piers	SHEET: A-3.90
ITEM:	OF
DESIGN: Pylons	REVISION:
DATE: IV 83 KM	

Resisting moment equals the sum of the moments about the compression face.

$$\phi M_N = (.9)\left[51500 - (4510)(4.55)/12\right] = 44811^{K-FT.}$$

$$44811 > 41856 \checkmark$$

Preliminary calcs. indicate the wall thickness must be increased

$$\phi v_c = (.85)(2)\sqrt{6000} = 131 \text{ psi}$$

$$\phi v_s = \frac{(2616)(2)}{(9952)(.85)} - .131 = .487$$

$$A_v/ft = \frac{(.487)(24)(12)}{60} = 2.34 \text{ in}^2/ft.$$

$$\frac{2.34}{2 \text{ sides}} = 1.17 \text{ in}^2/ft$$

or #6@6"

increase the wall thickness in the effective shear area only

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PROJECT: New Piers
ITEM: [illegible]
DESIGN: [illegible]
DATE: [illegible]

SHEET: A-2.41
OF
REVISION:

Pile Design:

Piles must resist overturning moment + shear
assume a max. 1-in-6 batter

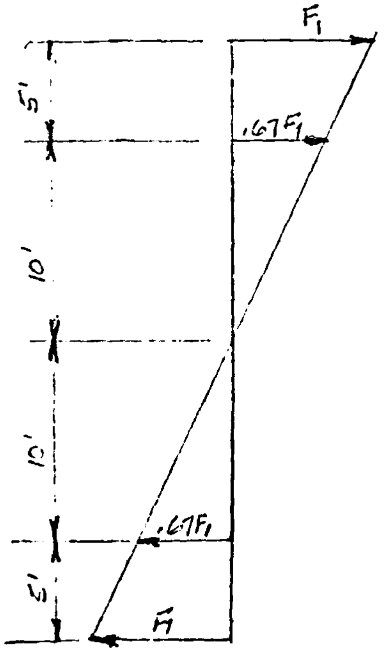
Shear

$V_u = 2616^k$

Overturning

$M_u = 20928^{k \cdot FT.}$

Since the force in the longitudinal direction is approx. 4 times that in the transverse direction, use a rectangular layout.



$30F_1 + (20)(.67F_1) = 41852$

$F_1 = 964^k$

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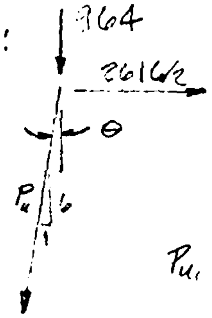
PROJECT: Navy Piers	SHEET: A-342
ITEM: Floating Mooring Pier, Stn. 2	OF
DESIGN: Pylon Foundation	REVISION:
DATE: IV 85 KM	

Pile Design:



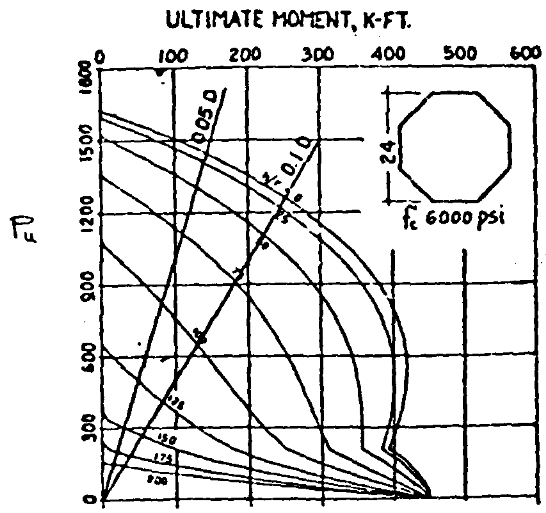
$\theta = \arctan(1/6)$

$\theta = 9.46°$

$$P_{u_1} = \frac{2616}{(2)(\sin 9.46)} + \frac{964}{\cos 9.46} = 8936^k$$

$$P_{u_2} = \frac{2616}{(2)(\sin 9.46)} + \frac{(.67)(964)}{\cos 9.46} = 8613^k$$

$P_{u\,allowable} \cong 1400^k$



$n_1 = \frac{8936}{1400} \cong 7$ piles

$n_2 = \frac{8613}{1400} = 7$ piles

assume allowable tension = 25% allowable compression

$n + .25n = 14$

$n =$ 12 piles on each side of the pylon

For the transverse direction use 4 piles on each side since the load is approx. 1/3.

Pylon/foundation interphase

6000 psi concrete; minimum 11.5' diameter pylon @ the lower end; 1-0" contact surface.

allowable compressive stress = (.7)(.85)(6000) = 3570
allowable force = (3570)(12)(138) ≅ 5912^k

moment arm = $\frac{41856}{5912} \cong 7.0'$

w/ 1'-0 cover on the bottom and 1'-0" to contact surface T&B.
total depth of foundation =

$$\begin{array}{r} +\ 7 \\ +\ 1' \\ +\ 1 \\ +\ 1 \\ \hline 10'\text{-}0" \end{array}$$

wt. of pylon
assume 40'-0 of hollow pylon 13' O.D. / 11' I.D
10'-0 solid 12.5'

Volume = $\pi(6.5^2 - 5.5^2)(40) + (\pi)(6.25)^2(10) = 2735.2\ ft^3$

wt = (2735.2)(.15) = 411^k
or 205.5 TONS

with increased wall thickness due to shear
wt. ≅ 300 TONS.

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PROJECT: Navy Piers
ITEM: Floating Marina Pier, Sch. 2
DESIGN: Weight
DATE: IV 83 RM

SHEET: A-2.44
OF
REVISION:

Weight of Structure : Scheme 2

$f'_c = 7000$ psi Normal-weight concrete.
$\gamma_c = 150$ PCF

Spine :	Area (F^2)	Length (F)	Volum. (F^3)	Wt (K)
Joint @ B.Slab	$7^\phi \times 3.5$	1000	2.45×10^4	3675
Cross-sect. JJ	836	1000	8.36×10^5	125400
Transv. wall	1×40	31×154	1.91×10^5	28644
Transv. Beams	$1.5\times\overline{8}$	24×154	4.44×10^4	6653
Vert. Beams	$1.5\times\overline{8}$	$24\times\overline{33}$	9.5×10^3	1426
				165798 K
Fingers :				
Trench	30	$6\times540'$	9.7×10^4	14580
Cross-Sect.	274	$6\times360'$	5.91×10^5	88650
X-X's	310	$6\times120'$	2.23×10^5	33480
	$\overline{380}$	$6\times60'$	1.37×10^5	20520
Δ	2500	6×2.17	3.26×10^4	4890
Transv. Walls	$1'\times28'$	$6\times8\times76.7'$	1.03×10^5	15450
	$1'\times\overline{35}'$	$6\times2\times76.7'$	3.22×10^4	4830
	$1.25'\times\overline{42}'$	$6\times2\times100'$	6.3×10^4	9450
Transv. Beams	$\overline{1.3}'\times\overline{3.75}'$	$6\times18\times77'$	4.05×10^4	6081
Vert. Beams	$1.25'\times\overline{4.75}'$	$6\times18\times33'$	2.12×10^4	3174

Total Concrete wt. Σ 366903 K $\times 1.05$ (misc.)

Topping (2" asphalt): $2/12 \times 418200^\phi \times 0.085$ = 5924 K
Mechanical @ Top Deck & Spine Int.: $0.100\times562200^\phi$ = 56220 K
Total Dead Load = 447392 K ←

Live Load :
@ 400 psf @ Main Deck = $0.400\times418200^\phi$ = 167280
@ 150 psf @ Inter. Deck = 0.150×144200 = 21630
@ 200 psf @ Bott. Deck = 0.200×418200 = 83640
Total Live load = 272550 K ←

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PROJECT:	Navy Piers
ITEM:	Floating Marina Pier Sch. 2
DESIGN:	Draft - Results
DATE:	1/83 RM
SHEET:	A-2.15
OF	
REVISION:	

Draft :

Dead load draft :

a) Spine pier draft : $\dfrac{Wt. \text{ in air}}{\gamma_{H_2O\ fresh} \times \text{Surface Area}}$

Surface area includes Spine + Area Finger attached to Spine

S.A. = 144000 + 31500 × 2
= 207000 ♯

Wt_{air} = Spine wt + 1/3 Finger wt. + Topp. + Mech.
= [165798 + 0.33 × 201000] 1.05 + 2933 + 35100
= 254107

Draft (normal wt.) = 254107/0.062 × 207000 = 20' ← Normal

Draft (Light wt.) = (254107 × .12/.15)/0.062 × 207000 = 16' ← Light wt.

b) Finger Draft : 360' × 90'

Draft (normal wt.) = $\dfrac{[2/3 \times 201000 + 2991 + 21120]}{0.062 \times 211200}$ = 12' ← Normal

Draft (Light wt.) = 12' × .12/.15 = 9.6' ← Light wt.

Must balast spine pier to match Fingers when joining the segments in water

	Spine (Ft) Freeboard	Finger (Ft) Freeboard	Δ (Ft) Freeboard	Ballast Spine (psF)
Normal wt. Concrete	(44 - 20) = 24	(30 - 12) = 18	6	372
Light wt. Concrete	28	20.4	7.6	471
Normal wt. Sp. + Light wt. Fg.	24	20.4	3.6	223

c) Total lighship draft :
Normal wt. = 447392/0.062 × 418200 = 17.3' ← Normal wt.

Light wt. = 17.3 × .12/.15 = 14' ← Light wt.

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Cont' Draft:

Live Load Draft = $\frac{272550^{K}}{0.062 \times 418200 \#}$ = 10.5' ← Live L. Draft

Total drafts =

$DL_{normal} + LL$ = 17.3 + 10.5 = 27.8' ← Total d. normal

$DL_{Light} + LL$ = 14 + 10.5 = 24.5' ← Total d Light wt

Vertical Center of Gravity H under Full Live load:
(Normal wt. concrete only)

Spine	D.L (K)	$\bar{y}_{L,TOP}$ (F)	$(DL)(\bar{y})$ (K-F)	Spine	LL (K)	$\bar{y}_{TOP}$ (F)	$(LL)(\bar{y})$ (K-F)
Joint	3675	42	154350	Int. Slab	21630	16	346080
Cross-Sect. + Wall + Bm.	162123	21.8	3534281	bott. Slab	2884	40	115360
Mech.	14420	16	230720				
Fingers							
Trench	14580	3.5	51030	Bott. Slab	54840	32	1760973
Cross-Sect.	88650	15.3	1356345				
	33480	18.8	629424				
	25410	21.8	553938				
Walls+Beams	38985	17	662745				
			7.17×10^6				2.2×10^6

$\Sigma\ DL(\bar{y}) + LL(\bar{y}) = 7.17 \times 10^6 + 2.2 \times 10^6 = 9.39 \times 10^6$ K-F

$H = (\Sigma\ DL(\bar{y}) + LL(\bar{y})) / \Sigma(DL + LL)$

$= 9.39 \times 10^6 / (366903 + 5924 + 56220 + 272550)$

= 13.4' from the top

Center of Buoyancy

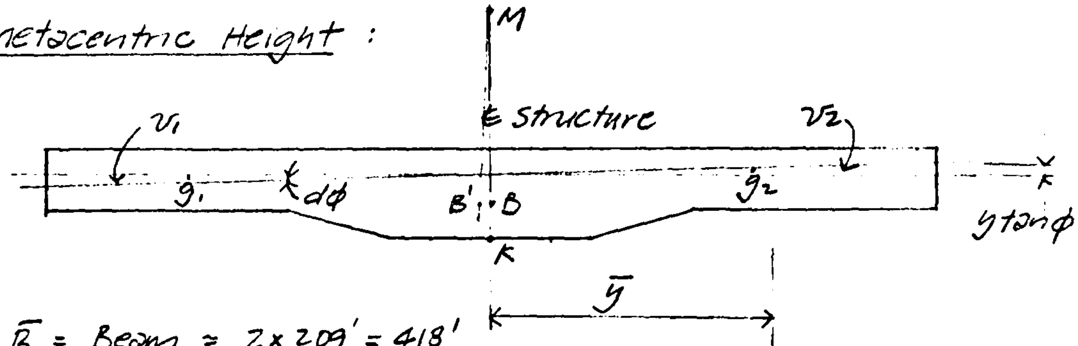
$\overline{KB}$ = (44' - 13.4')/2 = 15.3' above bottom slab

TYLIN INTERNATIONAL STRUCTURAL ENGINEERING 315 Bay St., San Francisco, Ca. 94133	PROJECT: Navy Piers ITEM: Floating Marine Pier, Sch. 2 DESIGN: Period x-x DATE: 1/83 KM	SHEET: A-247 OF REVISION:

Block Coefficient :

$C_b = 35\Delta / L \times B \times d = 35(701597) / 418200 \times 27.8 \times 2.24$
$= 0.94$

Metacentric Height :



$\bar{B}$ = Beam = $2 \times 209' = 418'$
$\bar{y} \cong (418200^{\phi} / 1000') \times 1/2$
$= 209'$

$\overline{BM} = \overline{BB_1} / \tan d\phi = v \times \overline{g_1 g_2} / \nabla \tan d\phi$ ∇ : Volumen of displacement $= 35\Delta$

For small ϕ :

$v = 1/2\ \bar{y}^2 \tan d\phi \times L$
$\nabla = L \times B \times d \times C_b = 418200 \times 27.8' \times 0.94$
$= 1.09 \times 10^7\ F^3$

$\overline{g_1 g_2} = 4/3\ \bar{y}$
$= 4/3 \times 209' = 278'$

$\overline{KB} = 15.3'$
$\overline{KM} = \overline{KB} + \overline{BM}$
$\overline{GM} = \overline{KM} - \overline{KG}$
$\overline{KG} = H = 30.6'$
$\phi = 10^\circ \rightarrow v = 1/2\,(209)^2 \tan 10^\circ \times 1000'$
$= 3.85 \times 10^6$

$\overline{BM} = 3.85 \times 10^6 \times 278 / 1.09 \times 10^7 \times \tan 10^\circ$
$= 557'$

$\overline{KM} = 15.3 + 557 = 572'$
$\overline{GM} = 572' - 30.6 = 541.4'$
$T\phi = C \times \bar{B} / \sqrt{GM} = 0.52 \times 418' / \sqrt{541} = 9.3$ sec ←

T.Y. Lin International Structural Engineering 315 Bay St., San Francisco, Ca 94133	PROJECT: Navy Piers	SHEET:
	ITEM: Floating Marina Pier, Sch. 2	OF
	DESIGN: Cost Estimate	REVISION:
	DATE: 10 33 KM	

Assume cost in-place concrete at $900/C.Y. (Ref. 1)

This cost includes: Concrete, Normal weight
Steel (Rebar + Tendons)
Piles
Ramp
Fenders
Construction — Flood Basin Method
Tow and Connection
Misc. @ 10%

Total C.Y. concrete $: (366903 \times 1.05)/(0.15^{K/ft}) \times 27$
$= 95123$ C.Y.

Total Concrete Cost (not including utilities)
$= 95123 \times 900$
$= \$85.6^{M}$

Utility cost by proportion From Ref. 1 :
$\$10^{M} - 168000^{\phi}$
$x - 418000^{\phi}$
$x = \$25^{M}$

Total cost of Pier (N.W.C.) $= 85.6 + 25$
$= 110.6$ Say $\underline{\$110^{M}}$ ← Normal wt. conc. pier

or $110 \times 10^6 / (418 \times 2 + 144) \times 10^3$
$= \underline{\$113/S.F}$ ←

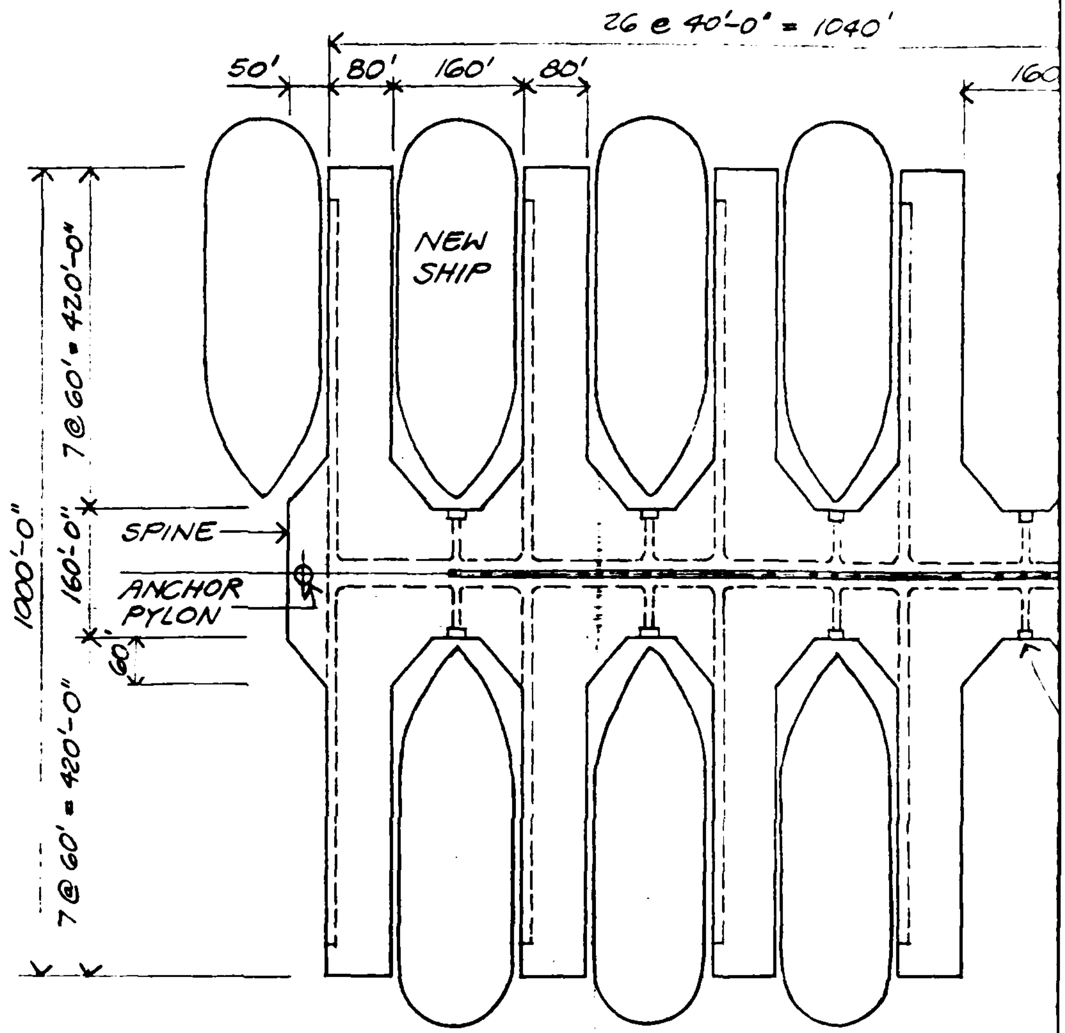
IF use Lightweight Concrete

Total cost Pier (L.W.C.) $= \$85^{M} \times \frac{1050}{900} \$/C.Y. + \$25^{M}$
$= \underline{\$124^{M}}$ ← Light wt. C.

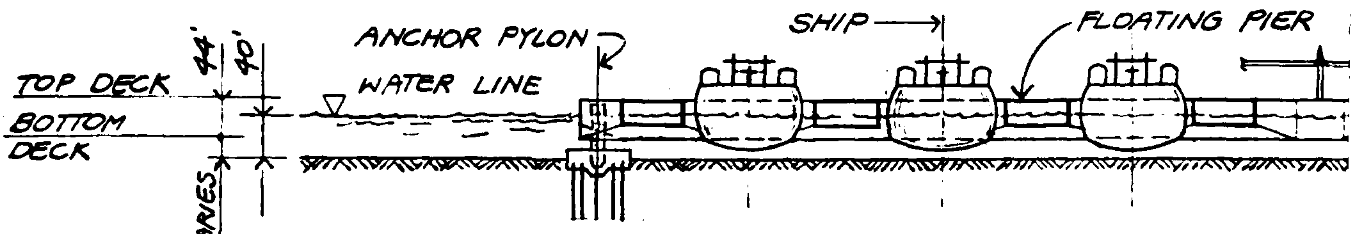
or $\underline{\$127/S.F}$ ←

(Ref. 1) By proportion From Conceptual design of Navy Floating Pier, June 1982 by T.Y. Lin International.

DRAFTING
DESIGN
PROJECT Nº



PLAN
1" = 200'-0"



ELEVATION
1" = 200'-0"

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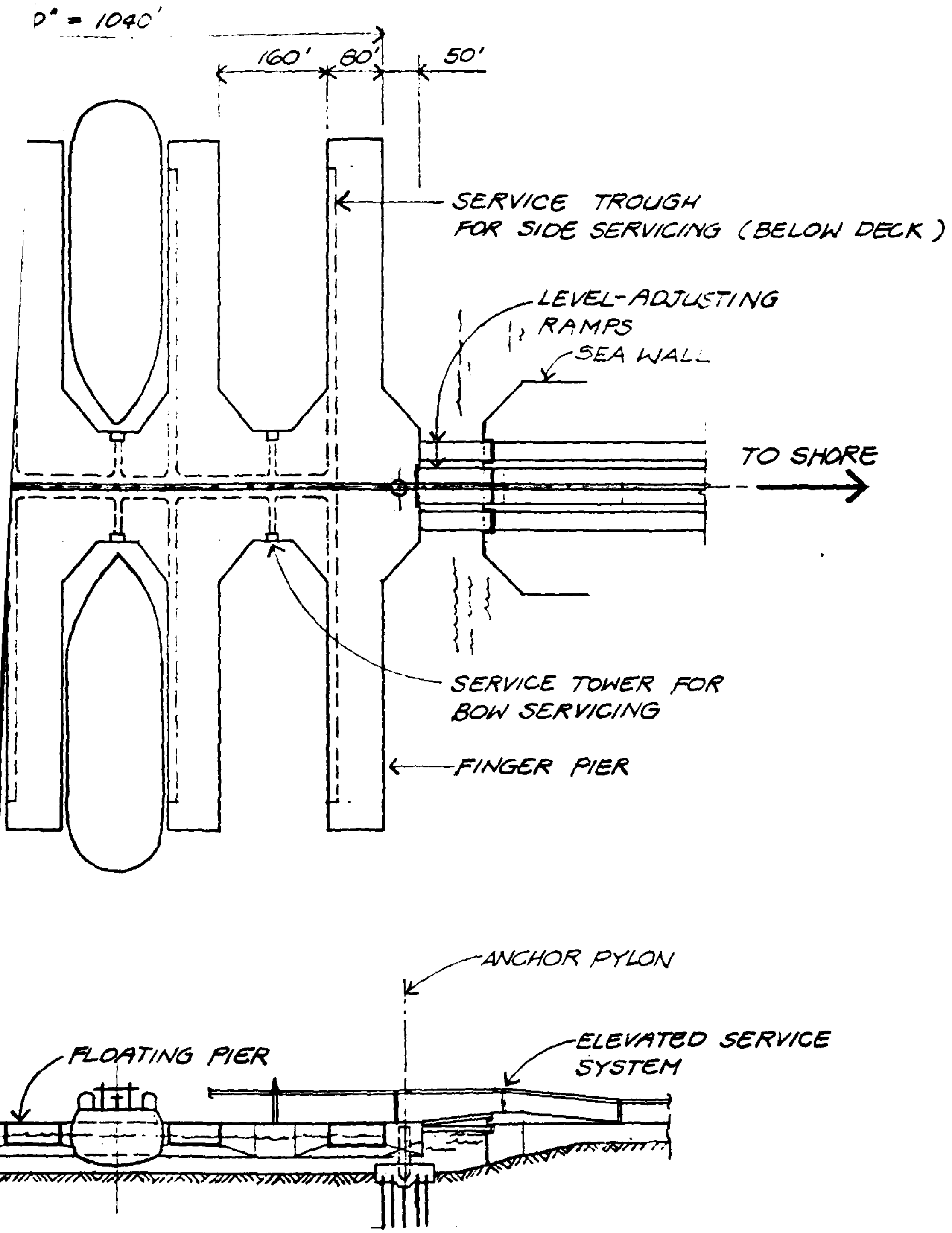


Nº	REVISION	DATE

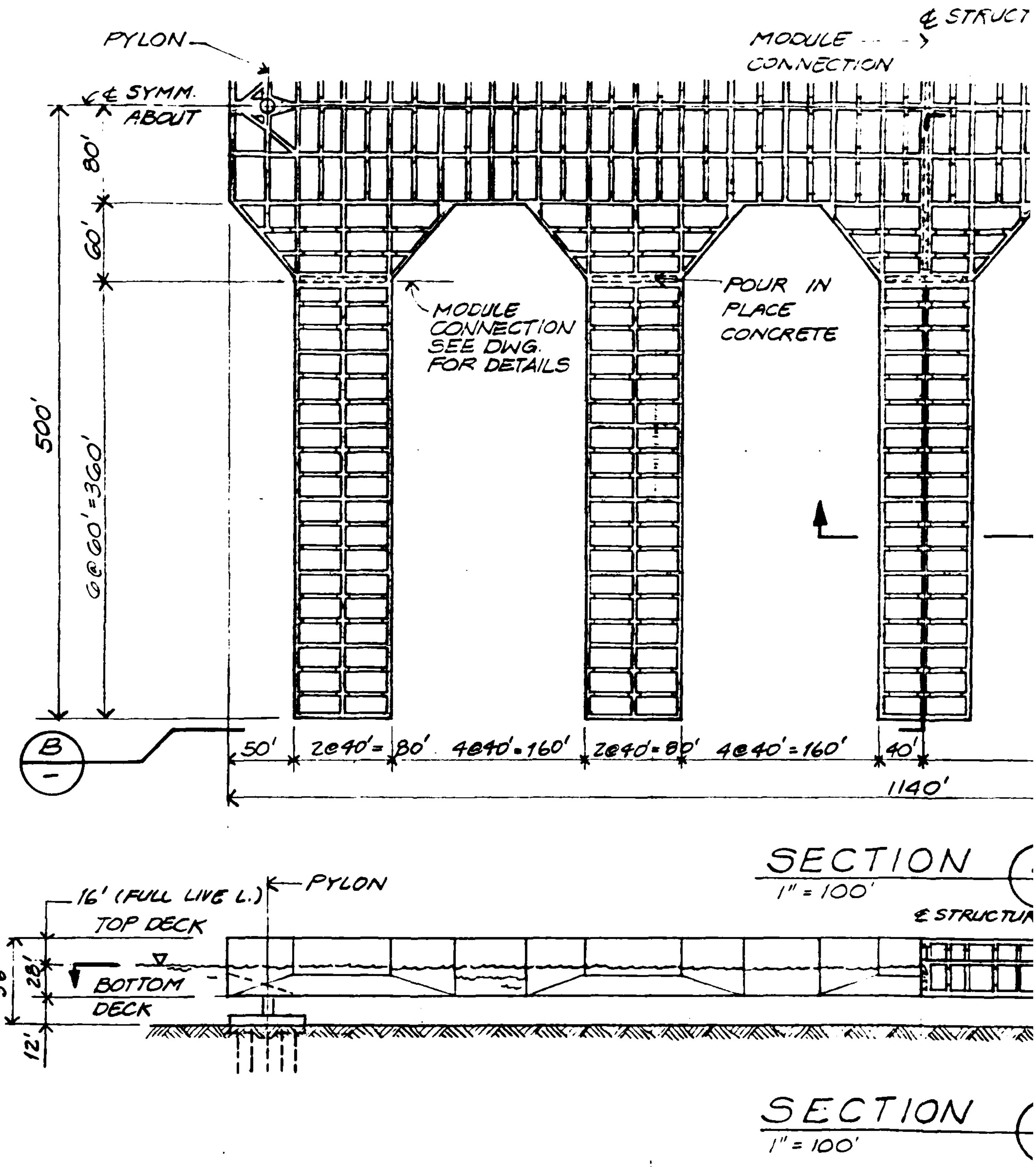
Issued For	Date	By
	Feb.83	RM

SHEET TITLE: PLAN AND ELEVATION - SCHEME 1

PROJECT: FLOATING MARINA PIER

2

SHEET Nº 1



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DESIGN

PROJECT Nº

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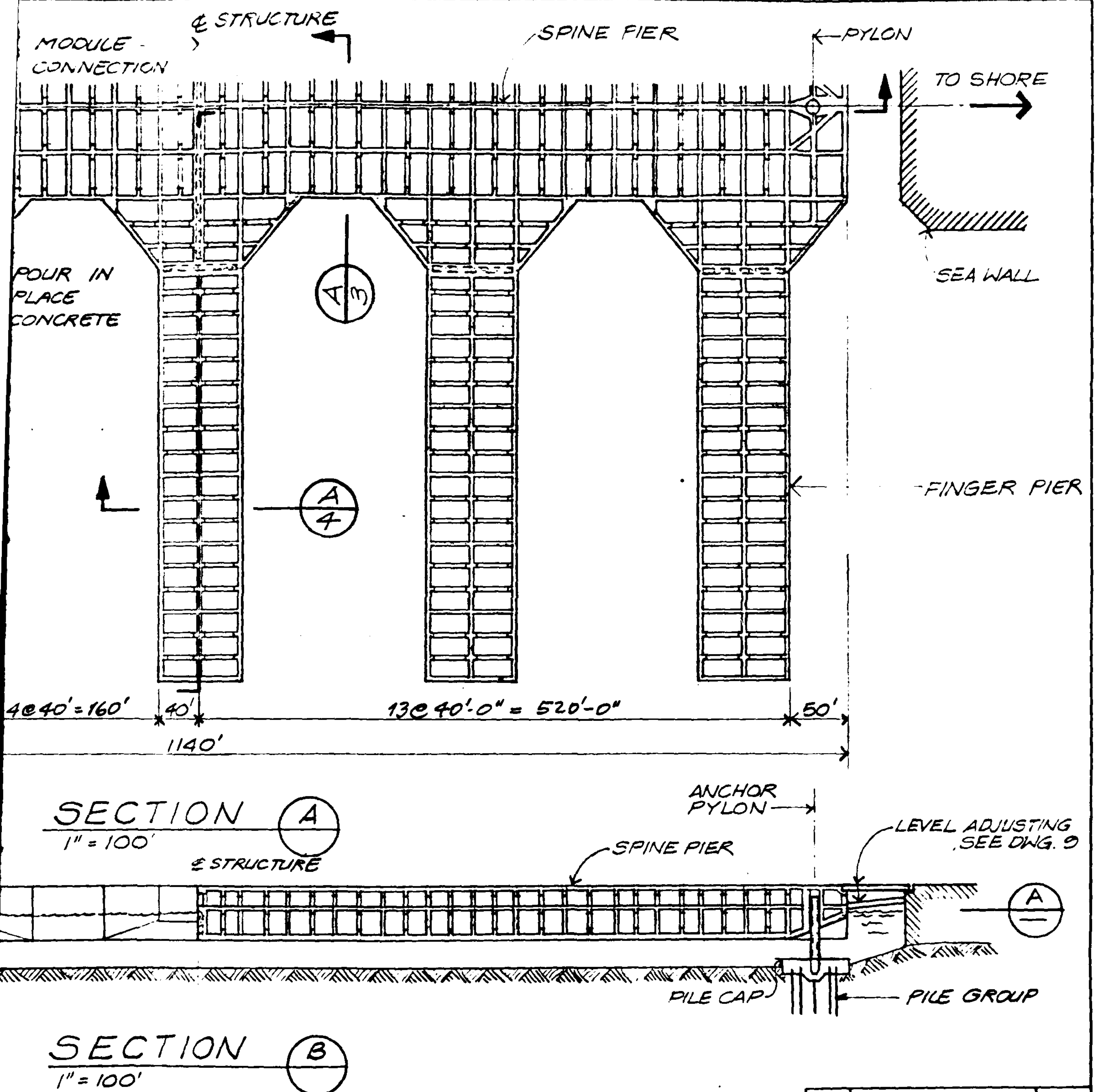
Issued For	Date



Nº	REVISION	DATE

Issued For	Date	By

SHEET TITLE: LONGITUDINAL SECTIONS - SCHEME 1

PROJECT: FLOATING MARINA PIER

SHEET Nº 2

PROJECT Nº

DESIGN

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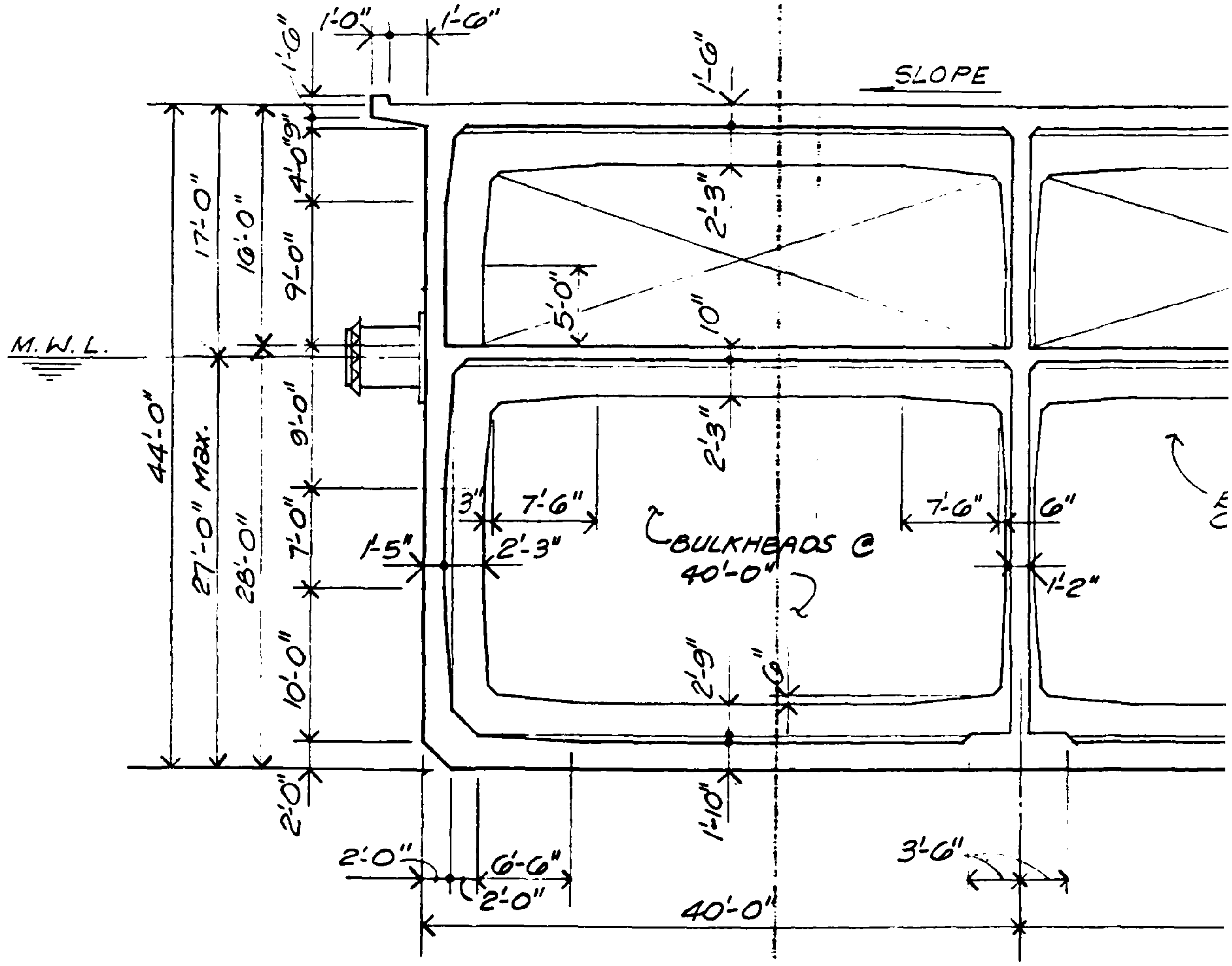

SECTION (A/3)

1" = 10'-0"

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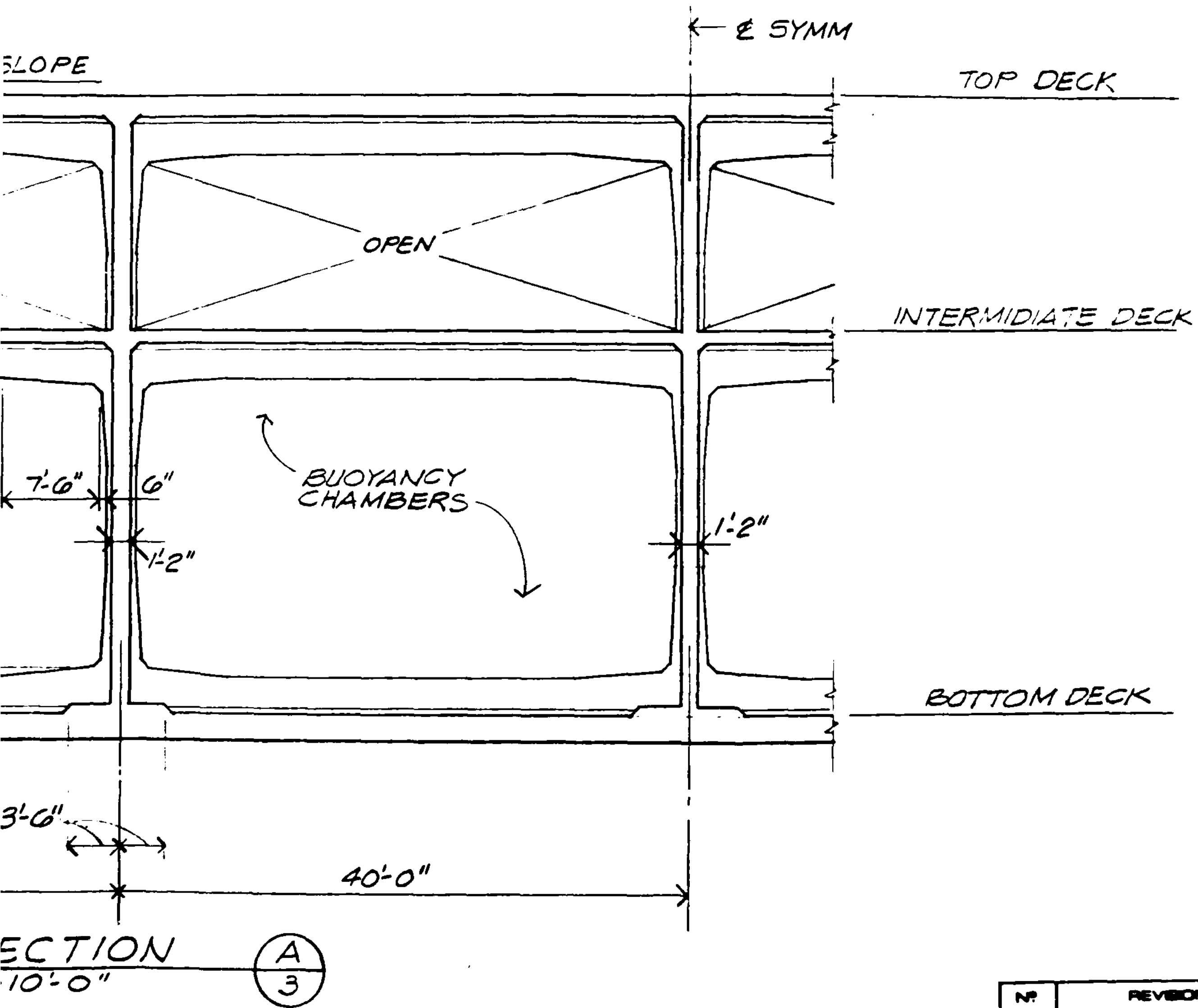

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SHEET TITLE: SPINE CROSS SECTION - SCHEME 1

PROJECT: FLOATING MARINA PIER

SHEET Nº 23

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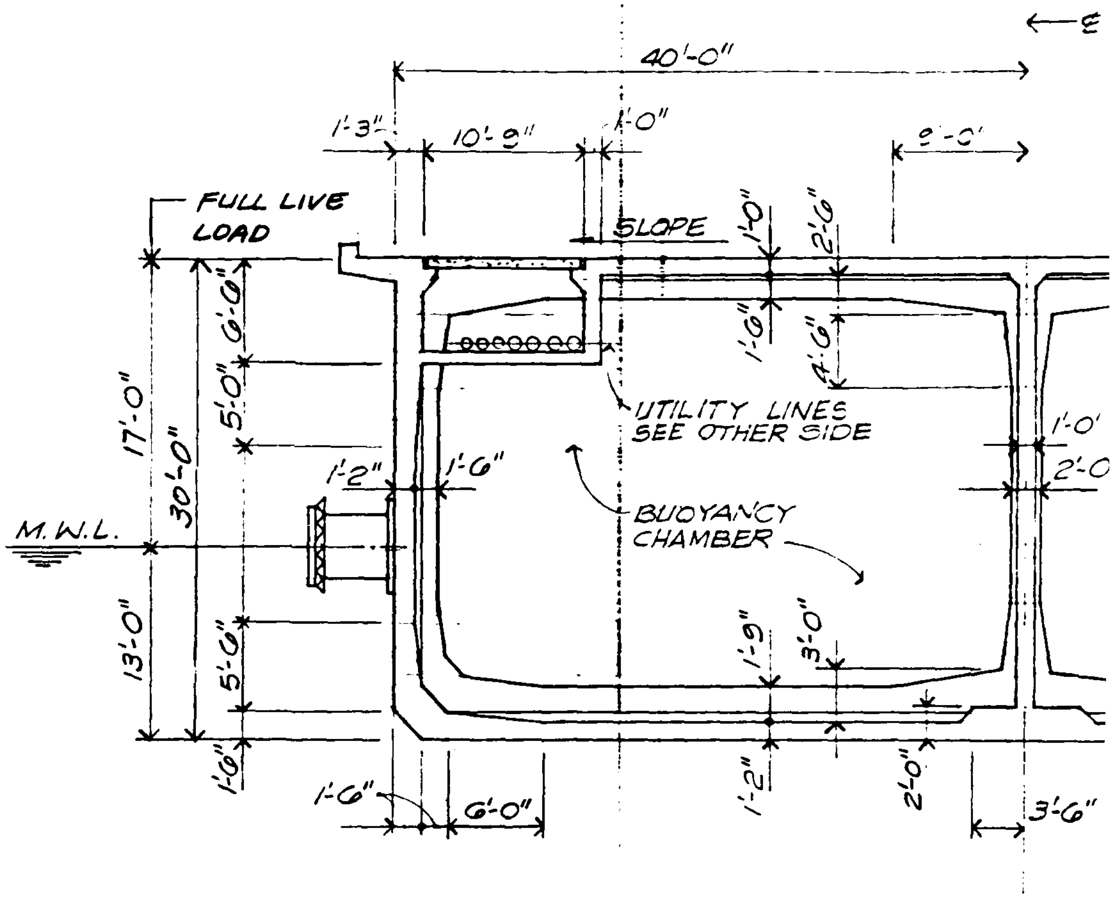

SECTION
1" = 10'-0"

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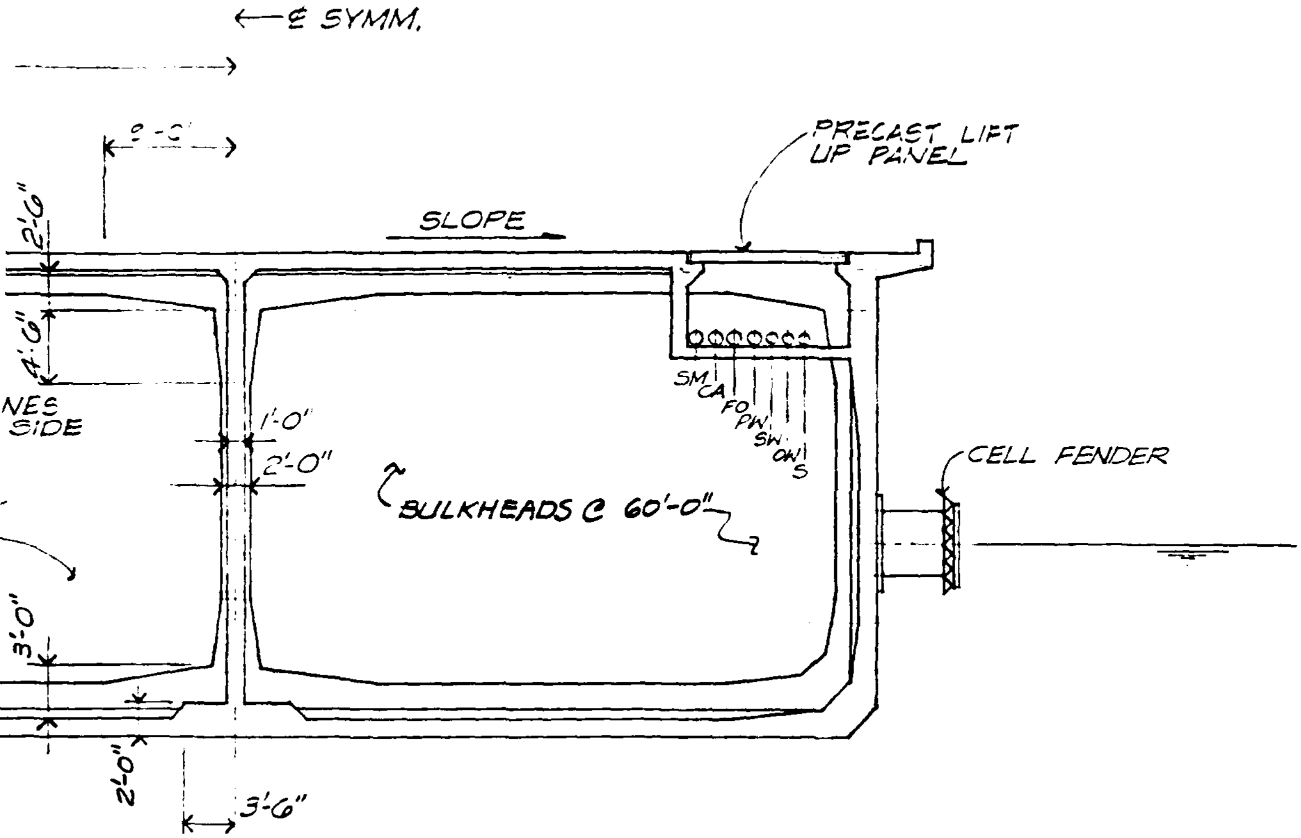

SECTION (A/4)
1" = 10'-0"

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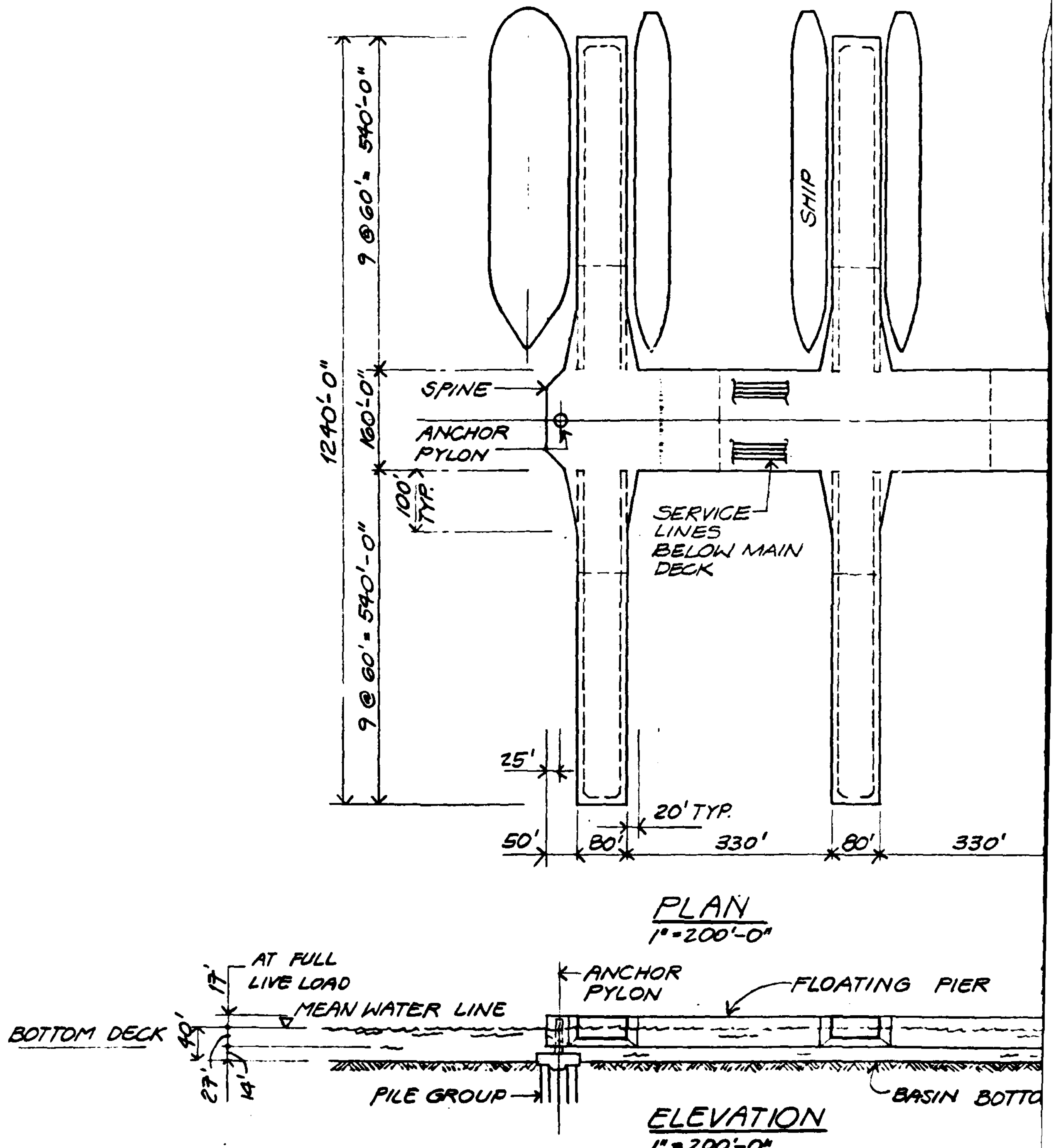
SHEET TITLE: FINGER CROSS SECTION - SCHEME 1 & 2

PROJECT: FLOATING MARINA PIER

SHEET Nº 4 2



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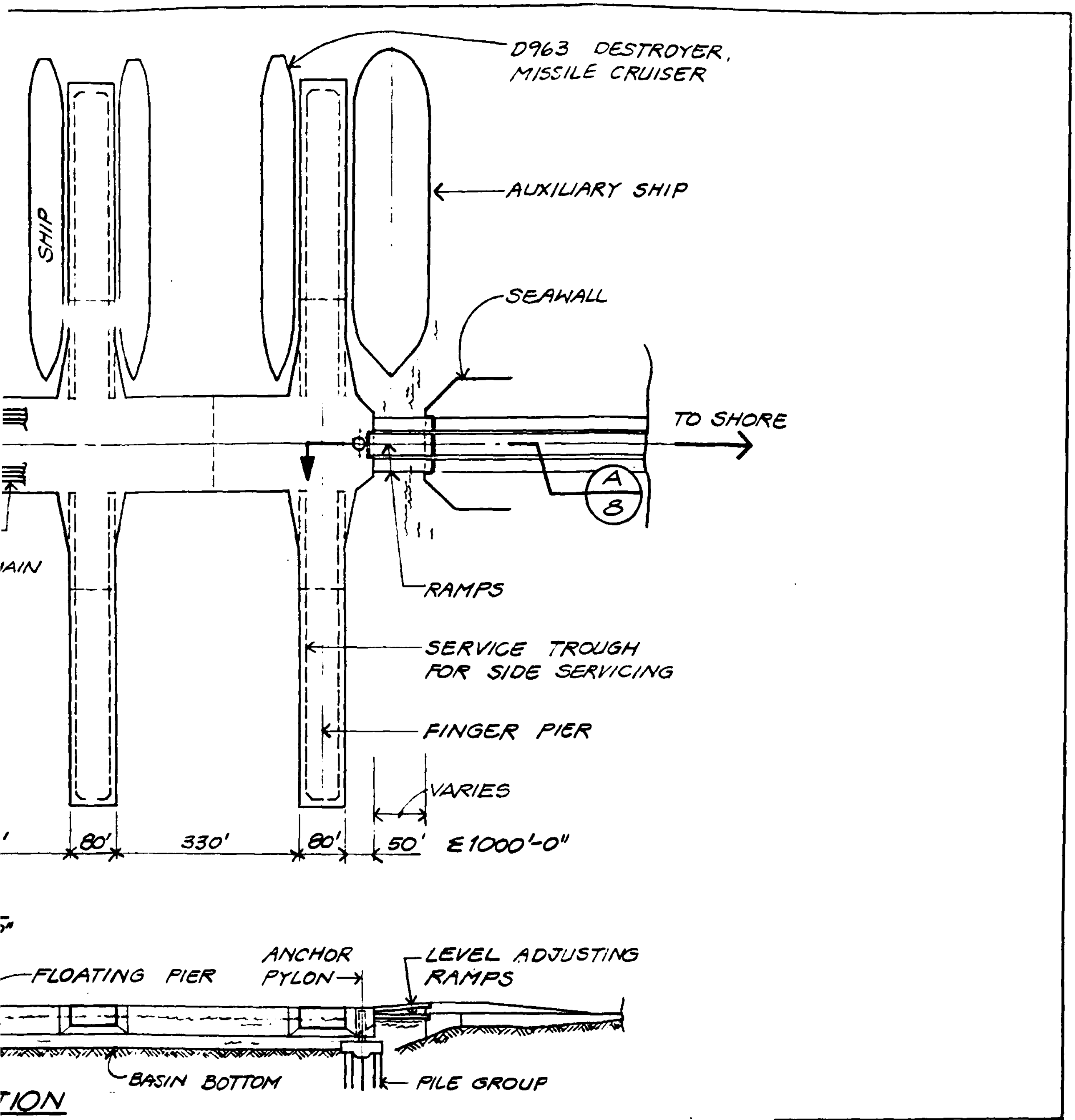

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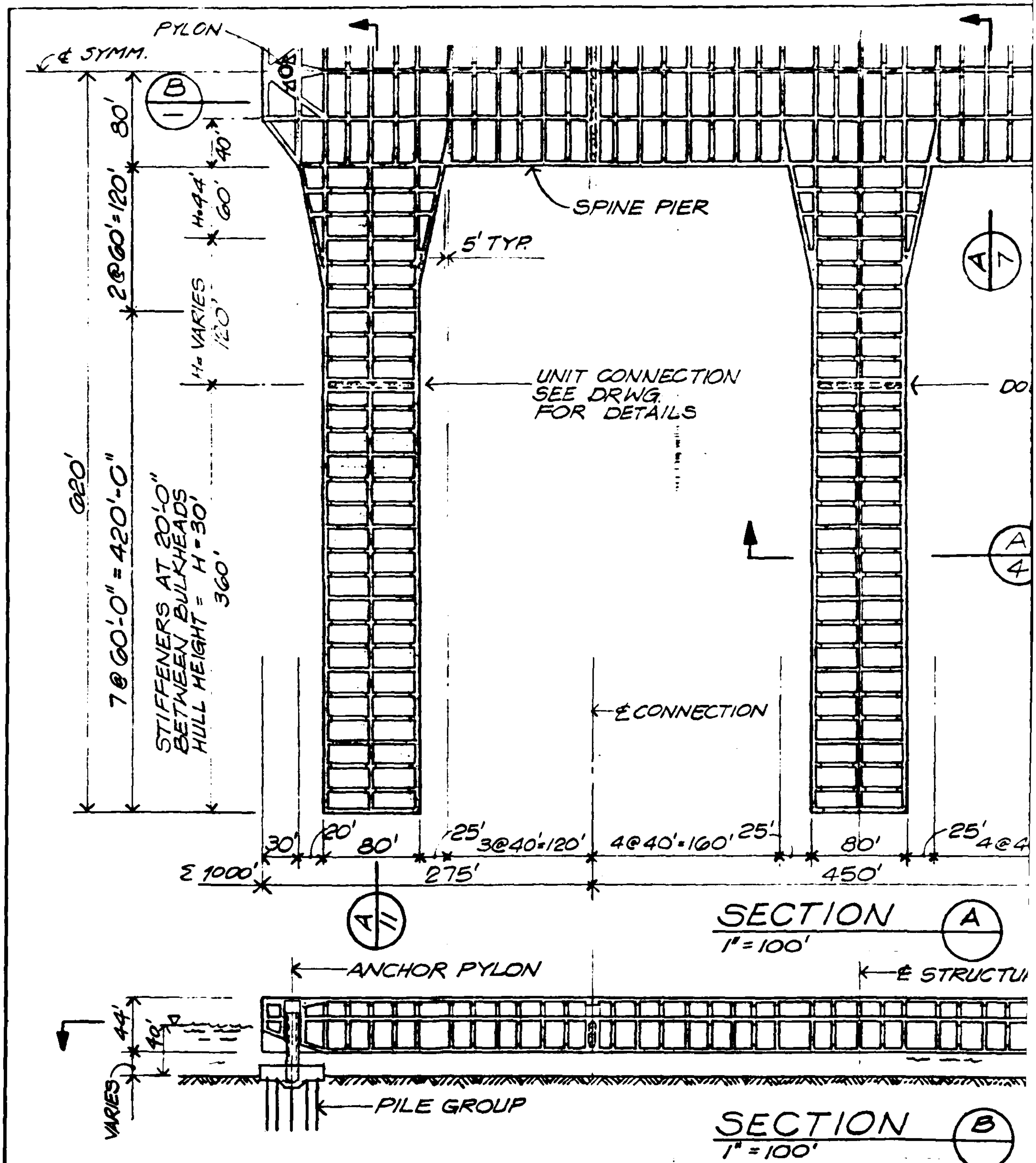
Issued For	Date	By
	Feb.83	RM

SHEET TITLE: PLAN AND ELEVATION - SCHEME 2

PROJECT: FLOATING MARINA PIER

SHEET Nº 5



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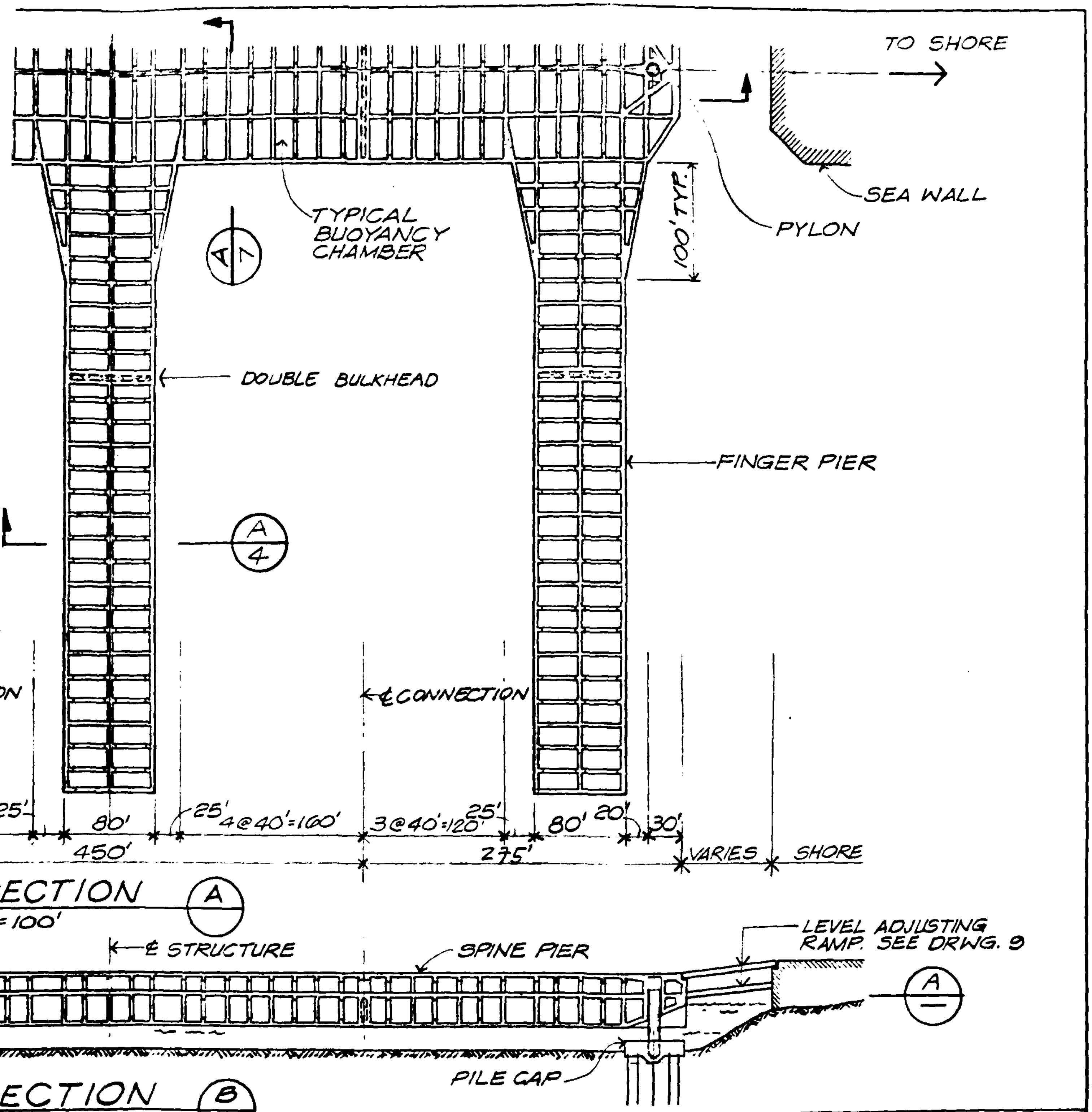

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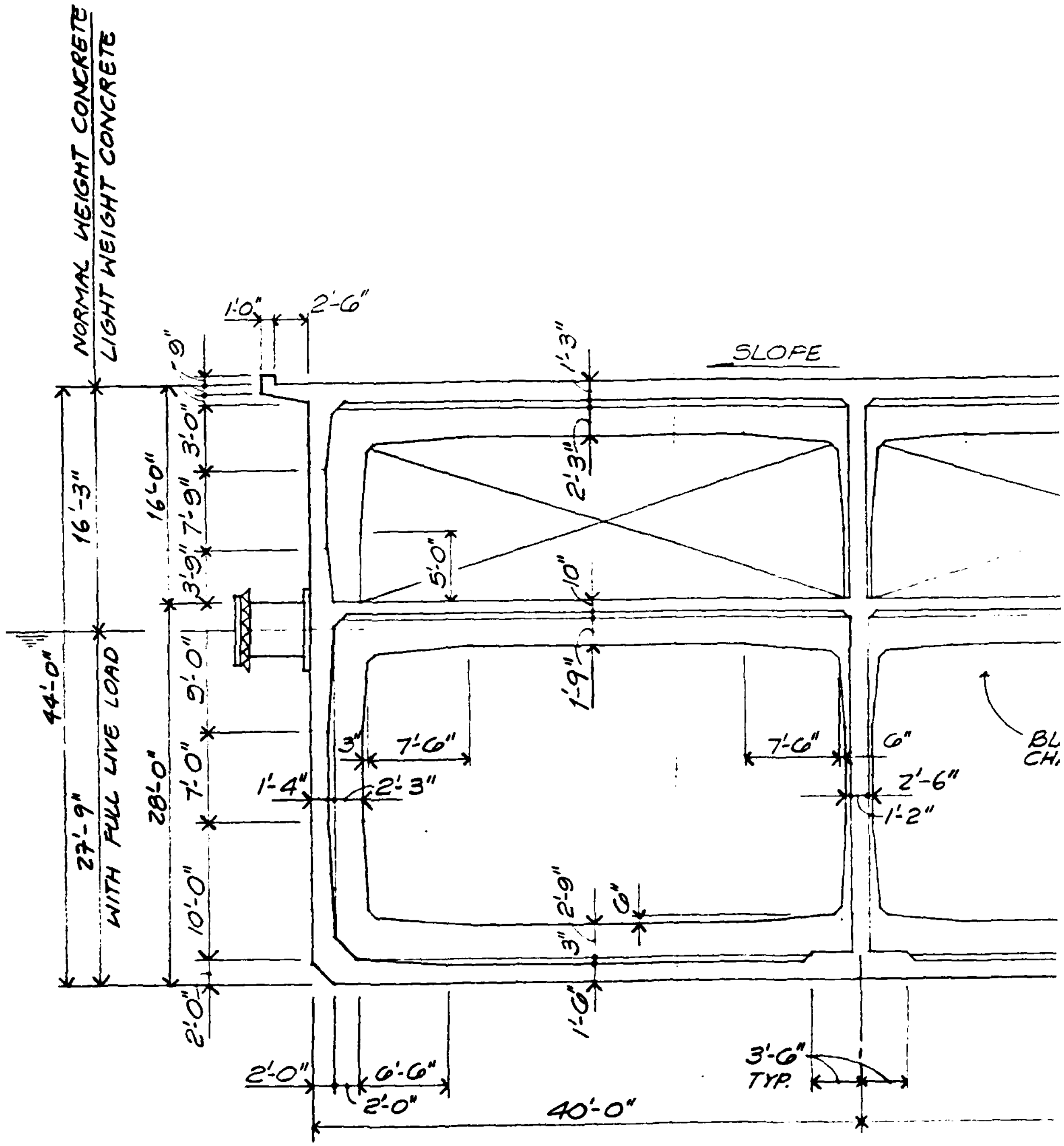
Issued For	Date	By
	Mar, 83	RM

SHEET TITLE: LONGITUDINAL SECTIONS - SCHEME 2

PROJECT: FLOATING MARINA PIER

SHEET Nº 2 6



SECTION (A/7)
1" = 10'-0"

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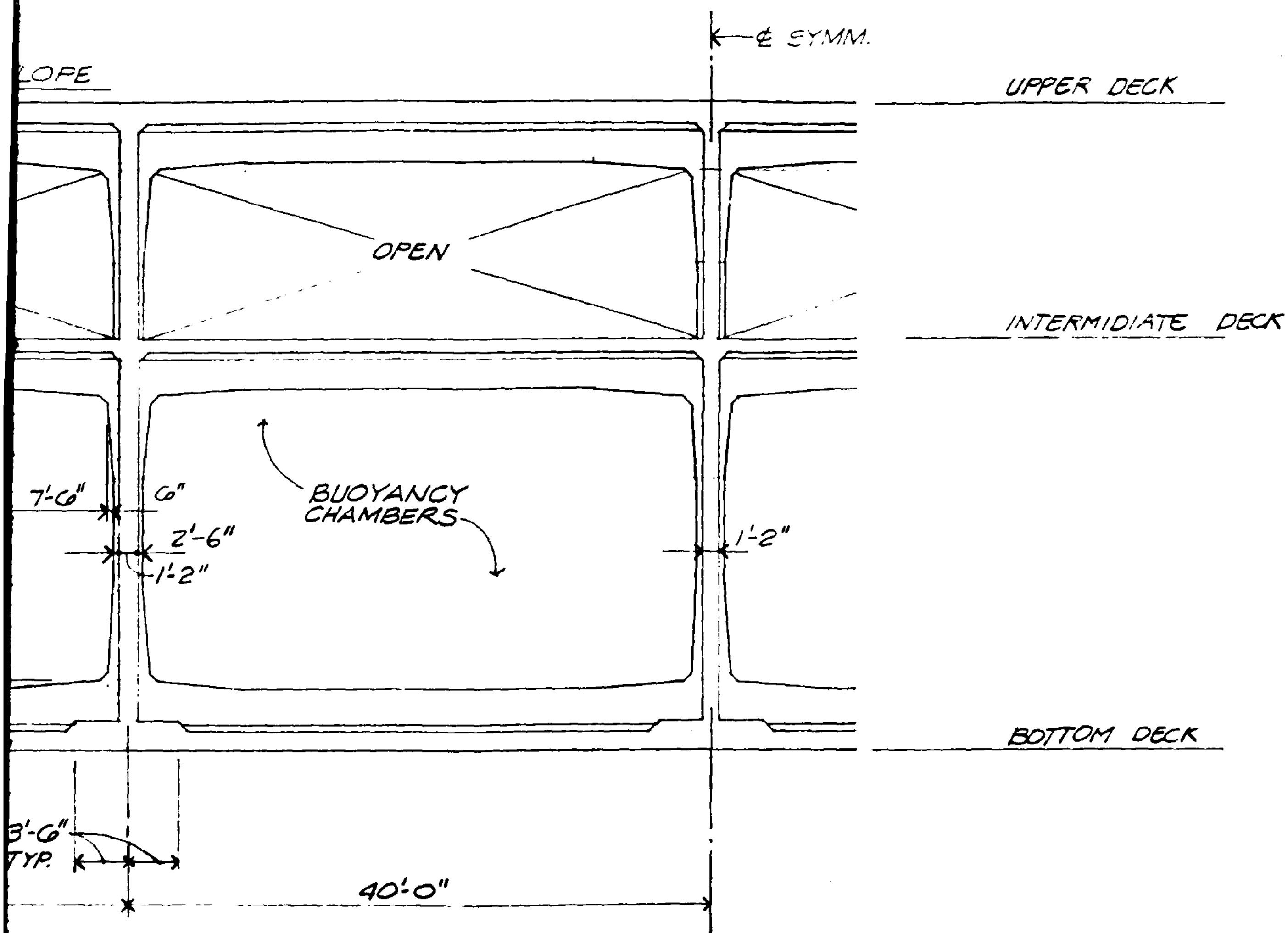

CTION (A/7)
/10'-0"

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SHEET TITLE: SPINE CROSS SECTION - SCHEME 2

PROJECT: FLOATING MARINA PIER

SHEET Nº 27

PROJECT Nº
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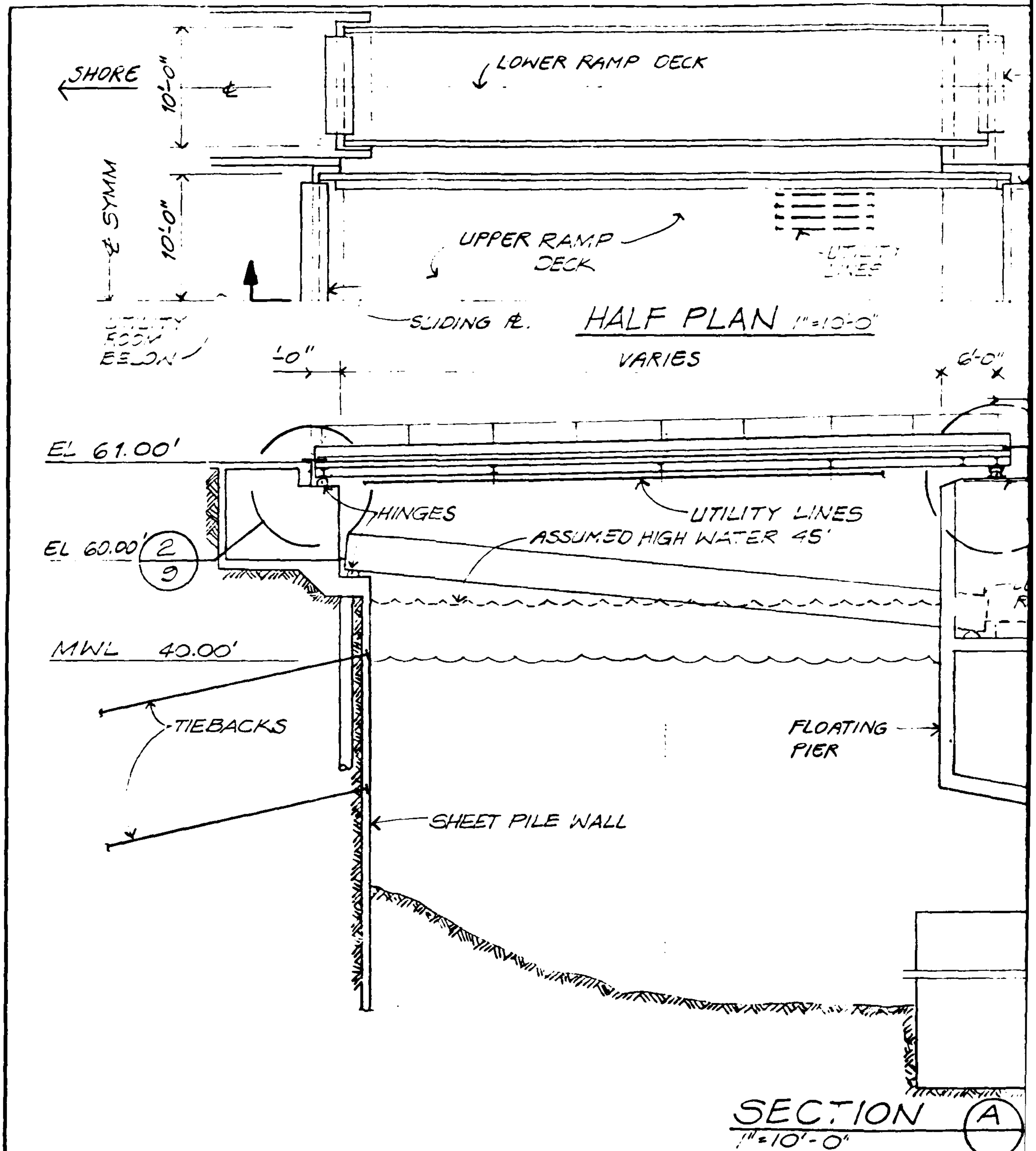

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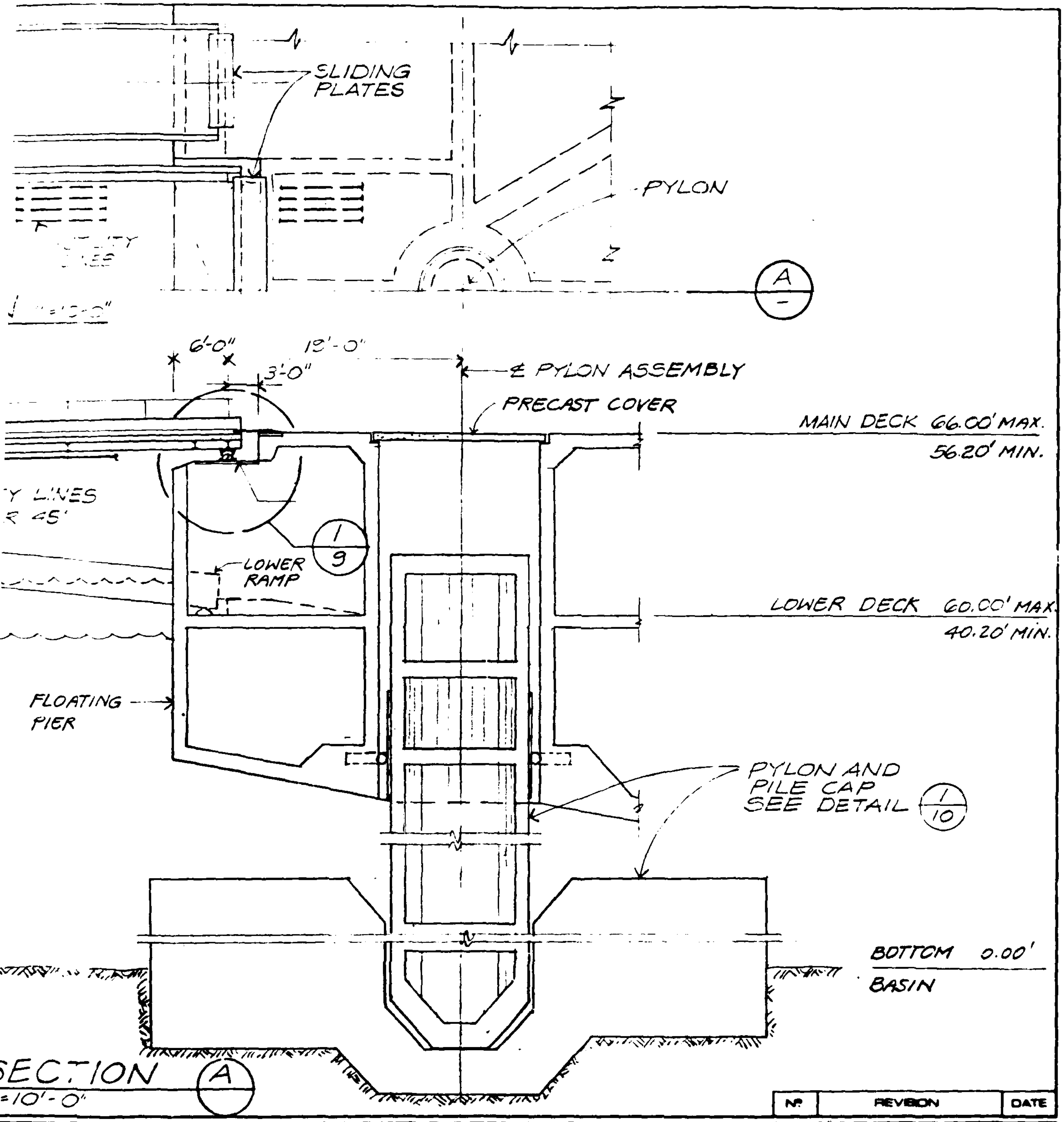

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SHEET TITLE: RAMP PLAN AND SECTION

PROJECT: FLOATING MARINA PIER

SHEET Nº 28

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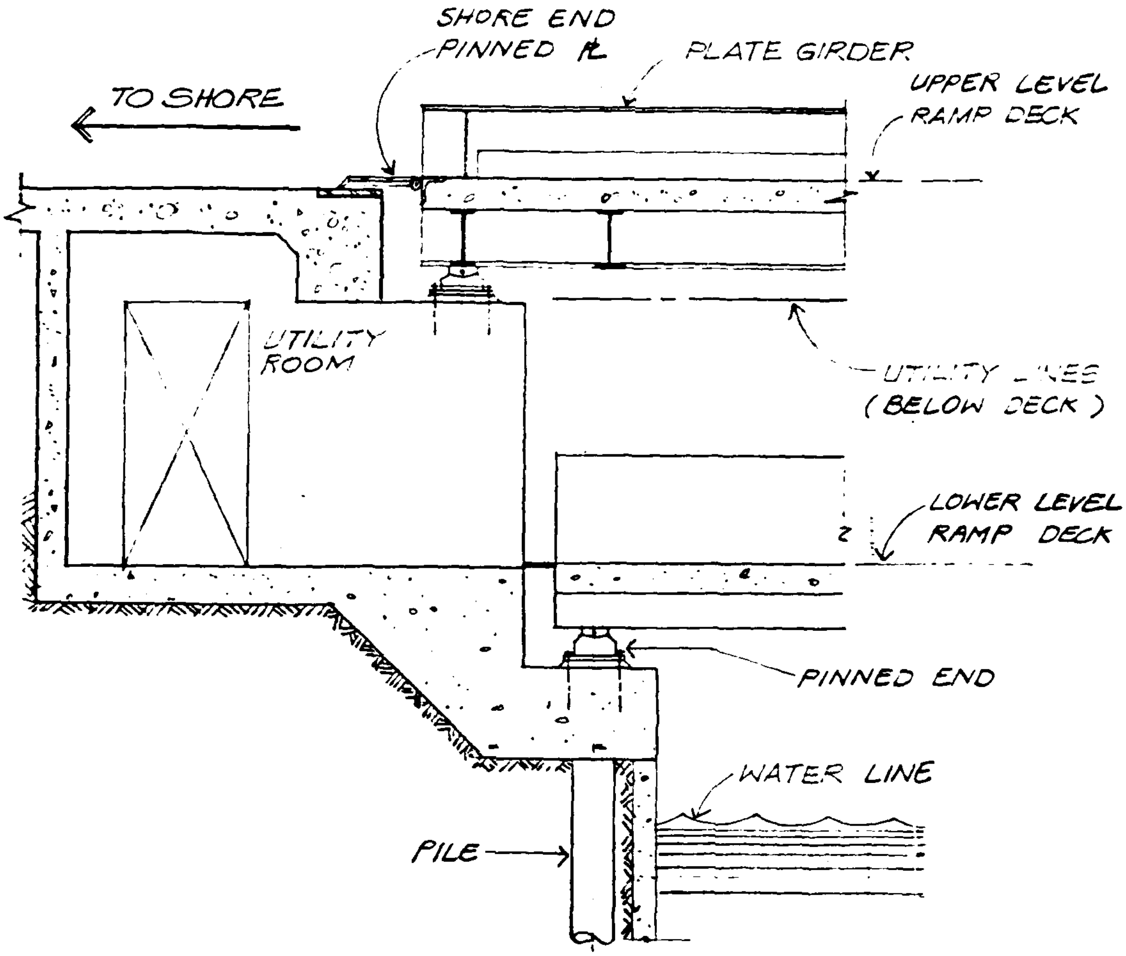

DETAIL 2

1/4" = 1'-0"

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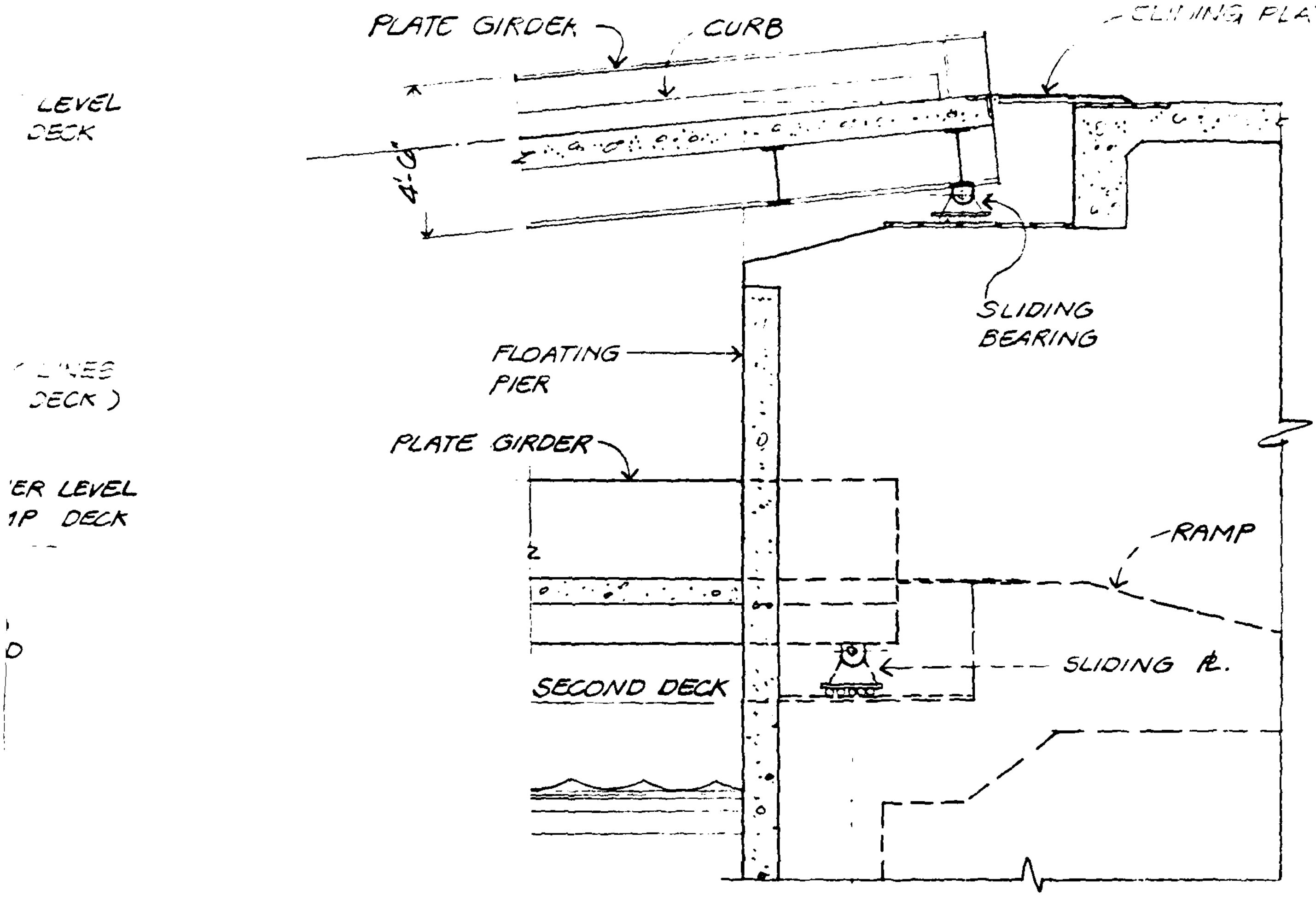

DETAIL 1
1/4" = 1'-0"

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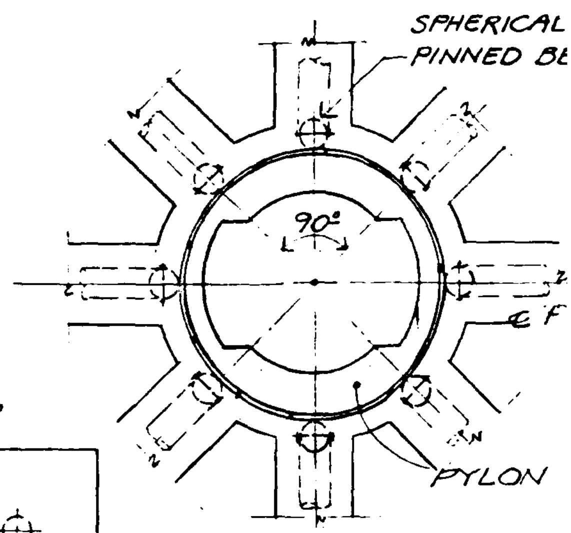
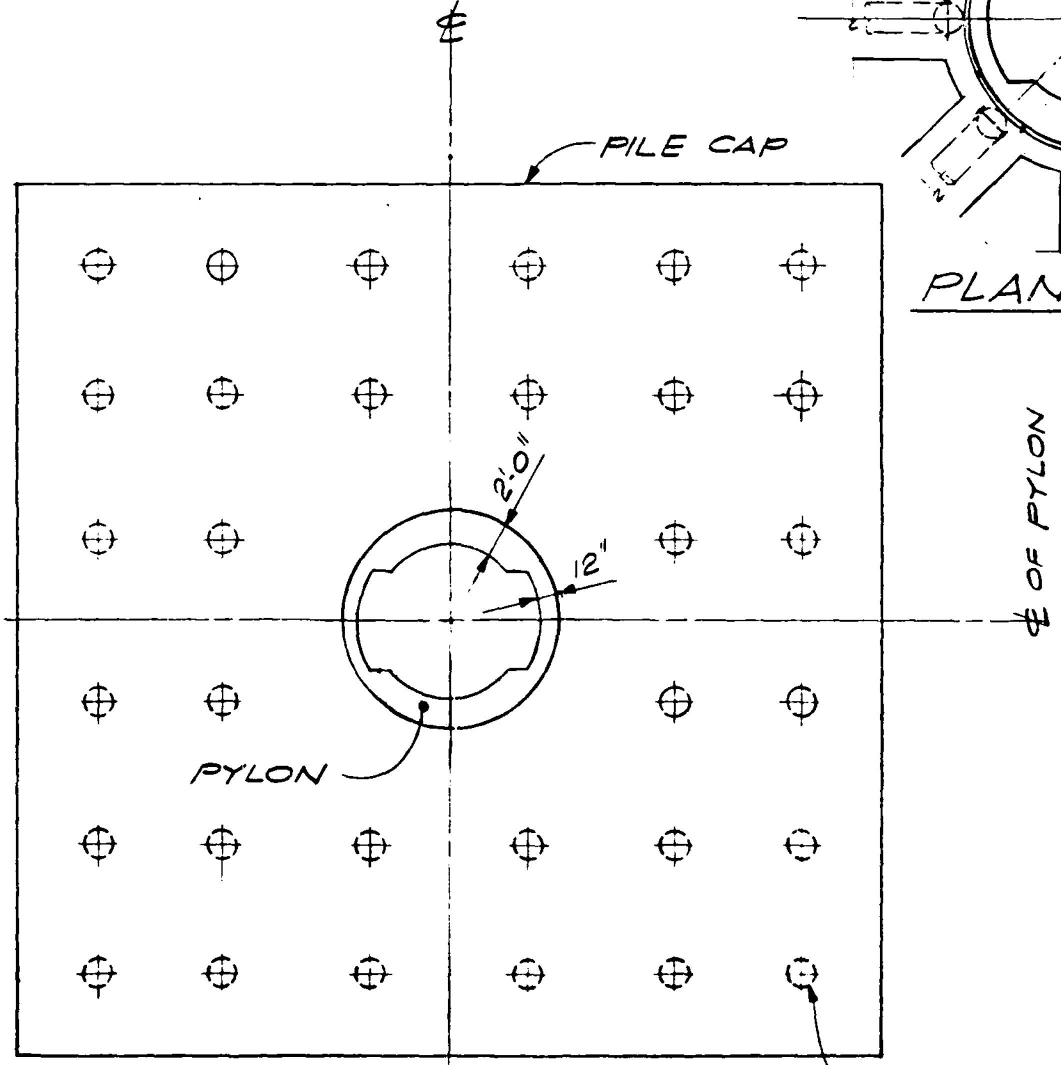
SHEET TITLE: RAMP CONNECTION DETAILS

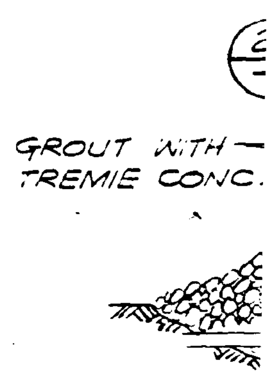
PROJECT: FLOATING MARINA PIER

SHEET Nº 29



PLAN SECTION





PILECAP PLAN 3

1" = 10'-0"

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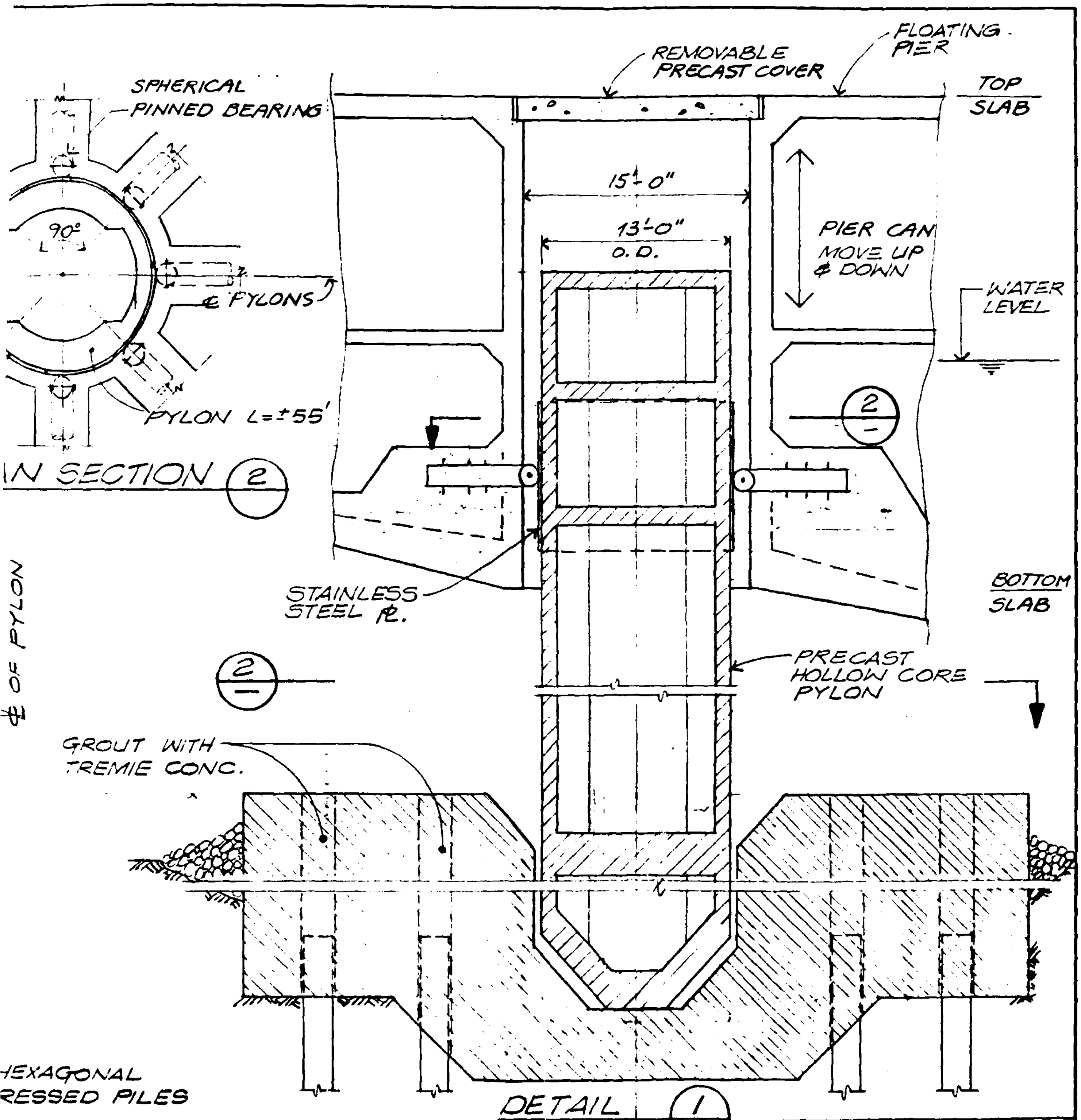

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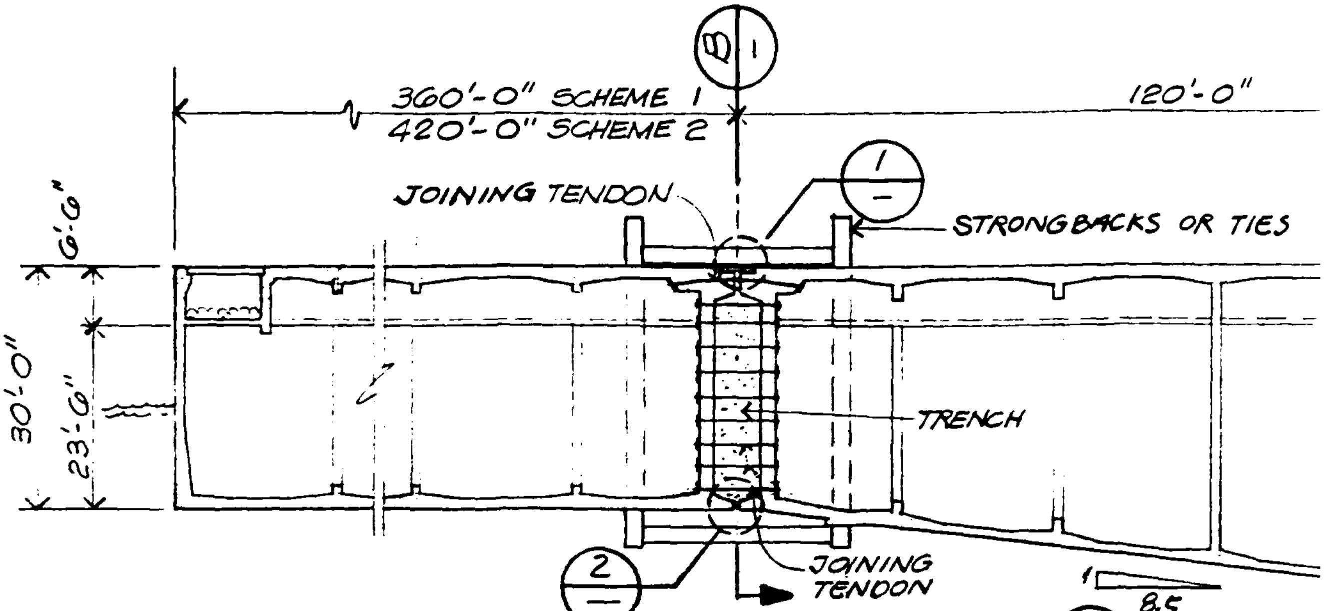
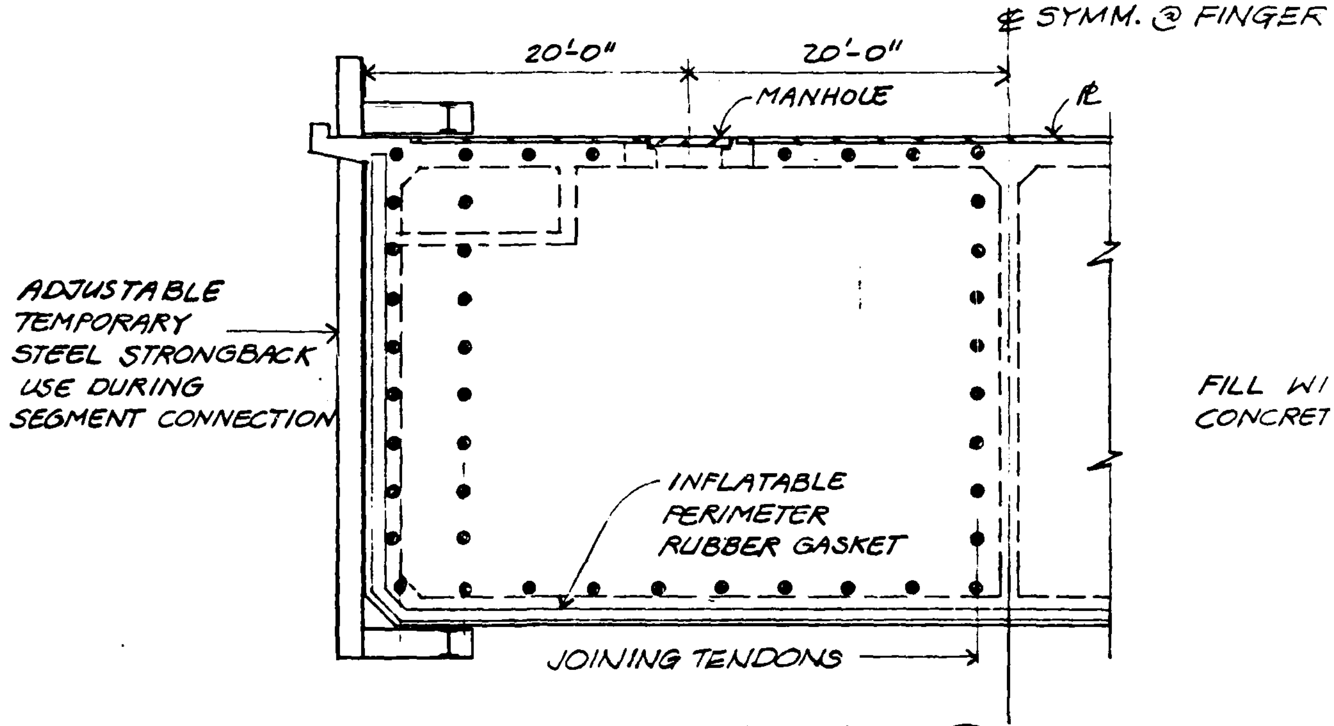
SHEET TITLE:	PYLON DETAILS
PROJECT:	FLOATING MARINA PIER
SHEET Nº	10



SECTION A
1" = 20'-0"



SECTION B
1" = 10'-0"

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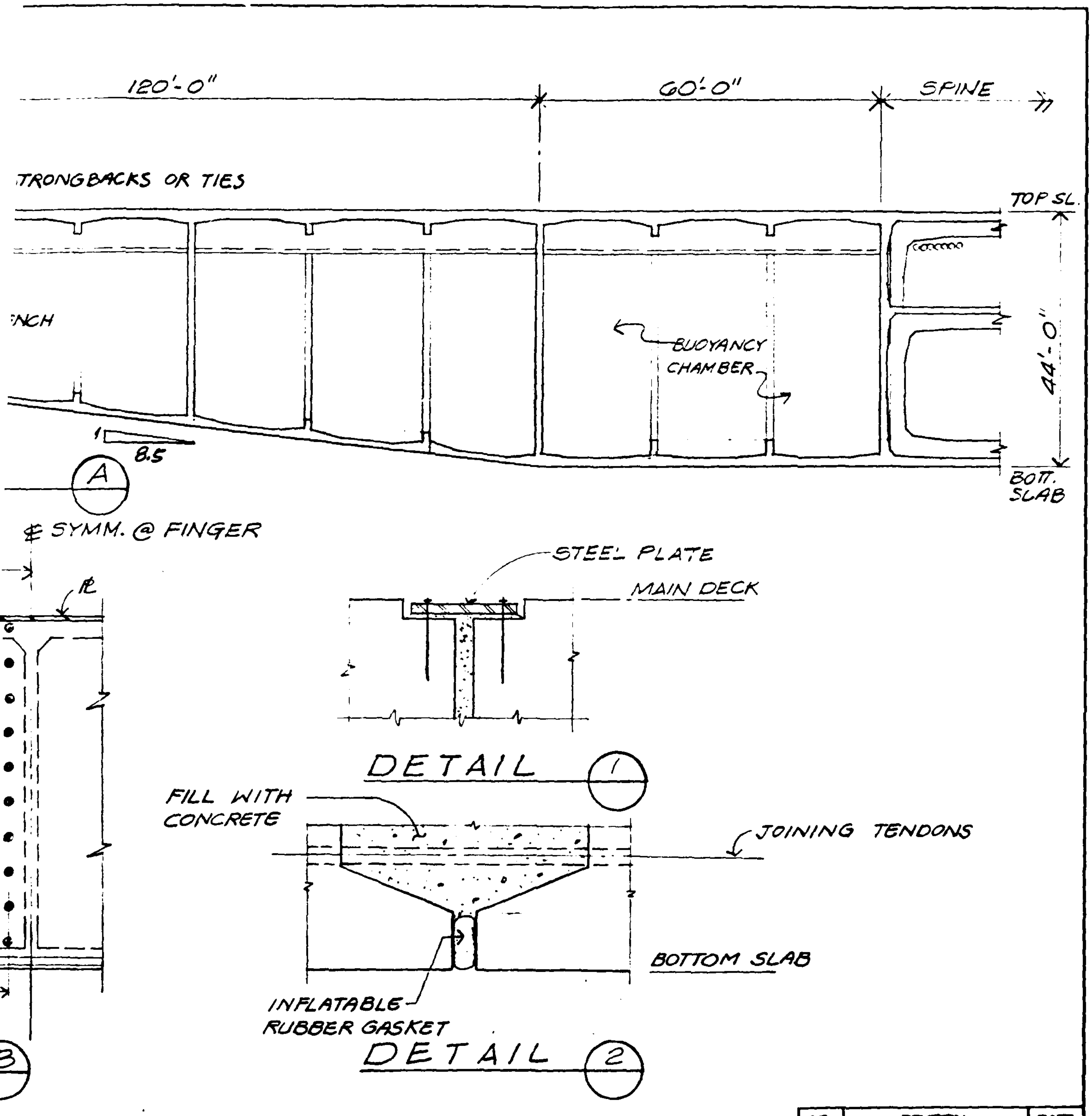

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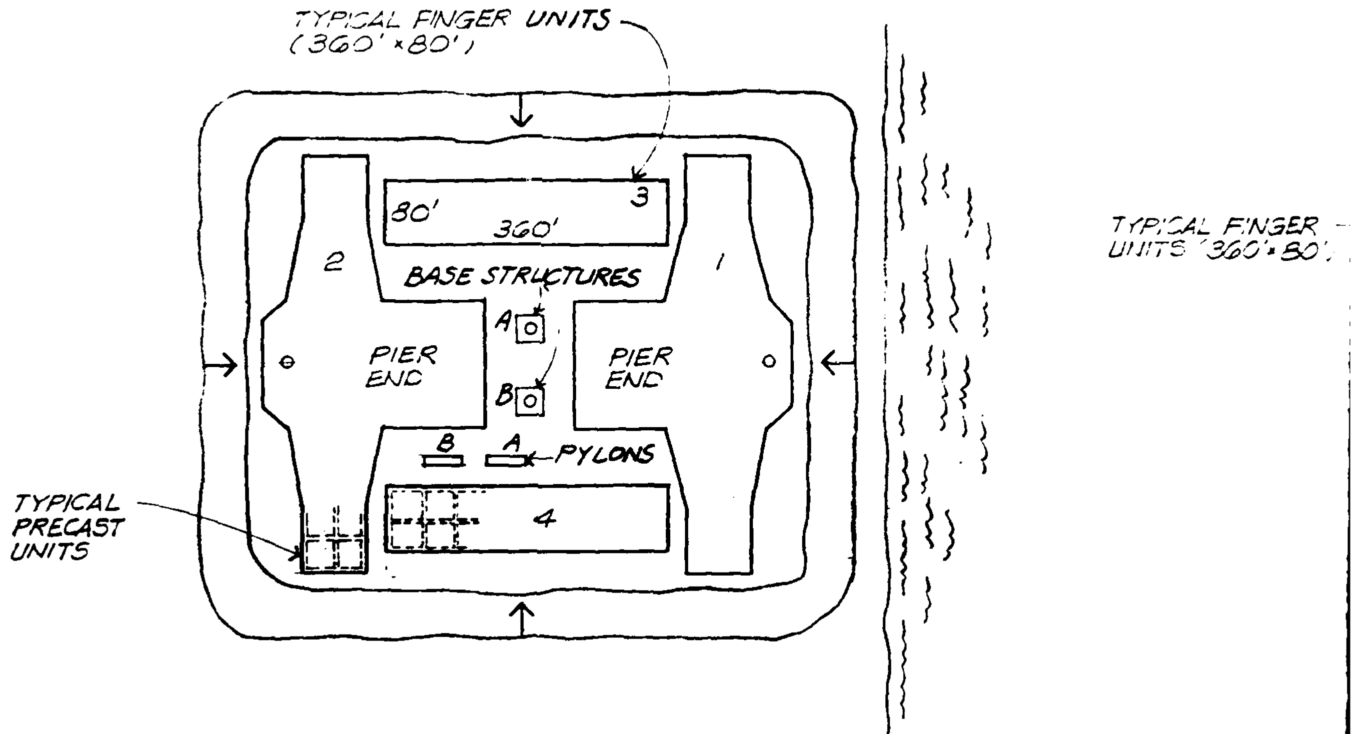
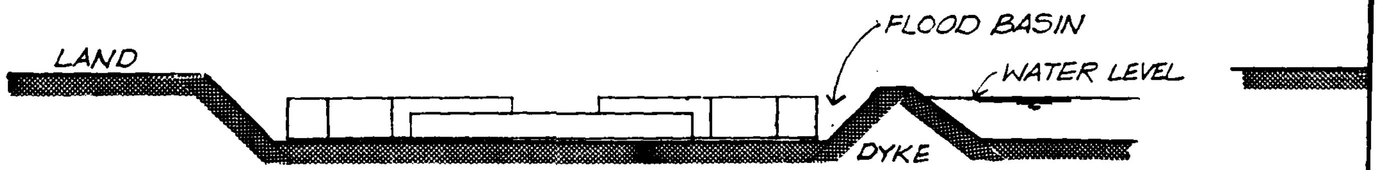
SHEET TITLE: MODULE CONNECTION

PROJECT: FLOATING MARINA PIER

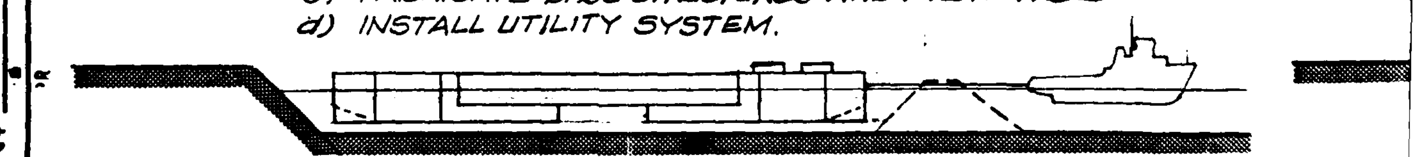
SHEET Nº 211





1. CONSTRUCTION IN FLOOD BASIN

a) BUILD ONE BASIN 700'× 580'.
b) JOIN PRECAST UNITS OF PIER ENDS (1, 2) AND FINGERS 3 AND 4 BY GROUTING AND POSTENSIONING.
c) FABRICATE BASE STRUCTURES AND PYLON A & B
d) INSTALL UTILITY SYSTEM.



2 TOW TO ASSEMBLY SITE

a) REMOVE DYKE TO FLOAT UNITS. FLOAT PYLONS ON PIER END 1
b) TOW TO FINAL SITE.

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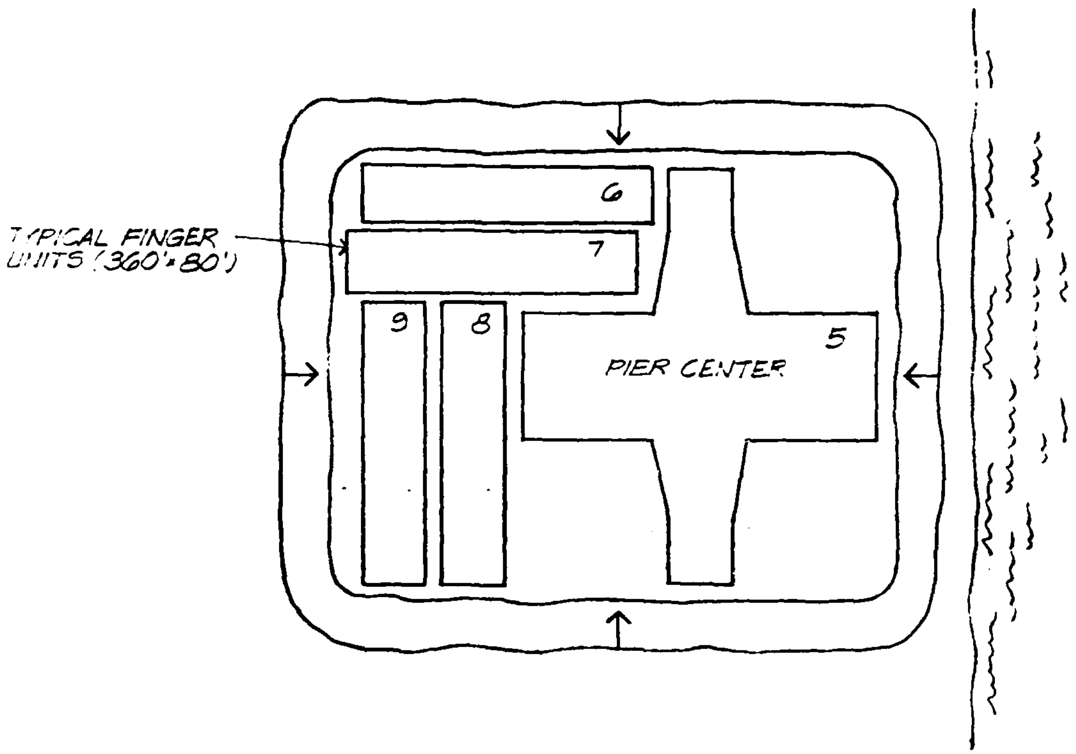
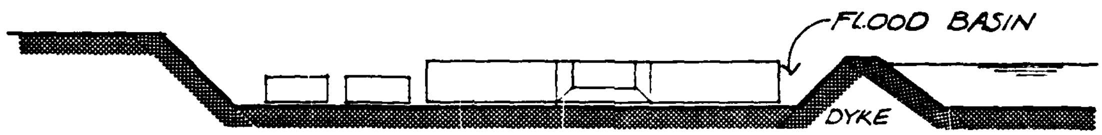


ASIN

WATER LEVEL

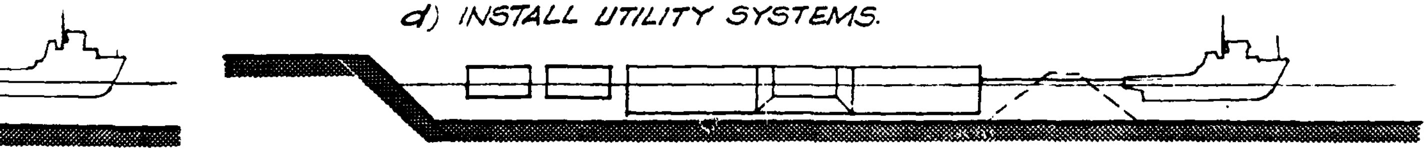


3. CONSTRUCTION IN FLOOD BASIN

a) REBUILD DYKE (USE SAME BASIN)
b) PUMP WATER OUT.
c) JOIN PRECAST UNITS OF PIER CENTER (5) AND FINGER UNITS 6, 7, 8 AND 9 BY GROUTING AND POSTENSIONING.
d) INSTALL UTILITY SYSTEMS.

ND FINGERS 3

. B



4 TOW TO ASSEMBLY SITE

a) REPEAT STEP 2.

ON PIER END 1

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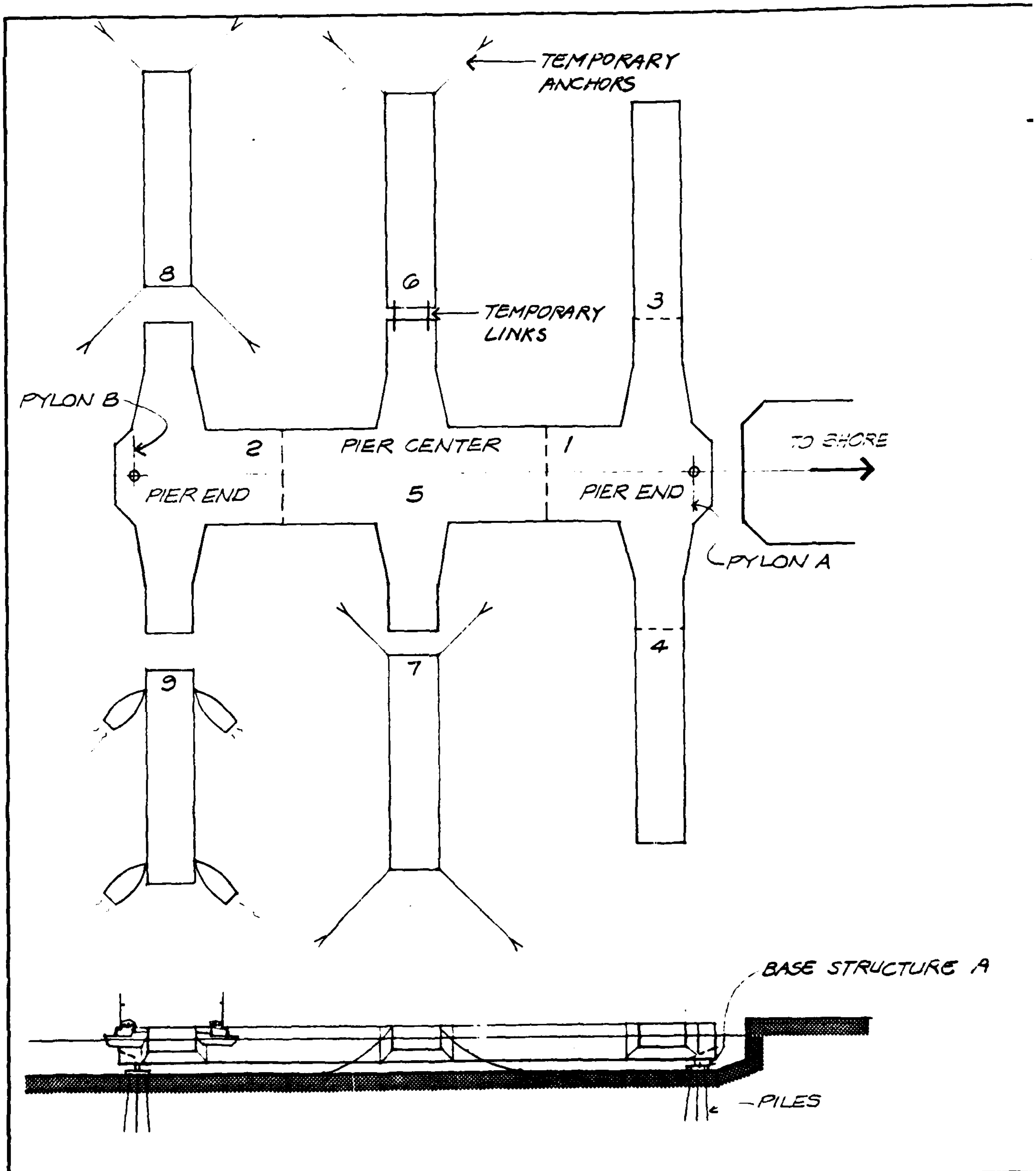
SHEET TITLE:	CONSTRUCTION METHOD - 1
PROJECT:	FLOATING MARINA PIER
SHEET Nº	12



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TO SHORE

N A

E STRUCTURE A

ES

5 INSTALL FLOATING MARINA PIER

a) SINK BASE ST.-A ON PREPARED BED. DRIVE PILES THROUGH OPENINGS AND GROUT PILE SLEEVES

b) POSITION PIER END 1 AT FINAL LOCATION AND INSTALL BY INSTALLING PYLON A

c) POSITION PIER CENTER 5 AND CONNECT TO PIER END 1 BY POSTENSIONING AND GROUTING.

d) POSITION PIER END 2 AND CONNECT TO PIER CENTER 5

e) SINK BASE ST. - B, DRIVE PILES AND GROUT PILE SLEEVES

f) INSTALL PYLON B

g) POSITION FINGER "3" IN FINAL LOCATION AND JOIN TO SPINE BY POSTENSIONING AND GROUTING.

h) REPEAT STEP g) FOR FINGERS 4, 6, 7, 8 AND 9.

i) INSTALL RAMP AND CONNECT UTILITY SYSTEM BETWEEN PIER UNITS AND SHORE.

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SHEET TITLE: CONSTRUCTION METHOD - 2

PROJECT: FLOATING MARINA PIER

SHEET Nº 13

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